LOAD SURVEY

of

A REINFORCED CONCRETE WAREHOUSE

A Thesis

Submitted for the Degree of

MASTER OF SCIENCE

By

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PREFACE

The warehouse surveyed in this thesis was built about April, 1907, by the Southern Ferro Concrete Company of Atlanta, Georgia. It was one of the first reinforced concrete structures to be built in that City and is a remarkably brave venture in building construction when we think of the inadequacy of knowledge on reinforced concrete at the time it was built.

To better show the magnitude of this last statement, a short but rather complete history of the growth of the theory of concrete design was undertaken. We feel that too few engineers realize the slow and laborous process of study and experiment that precede the establishment of a few fundamental principles of design so perhaps a resume of the many attempts at truth will impress them with the respect due to the pioneers in concrete design.

Of course, the first constructor has always preceded the first designer. We had wood, stone and iron long before Newton proclaimed his three fundamental laws of mechanics. So, for this reason, we have prefaced our work with a history of the making of concrete from antiquity to the present day, hoping that this will enlighten a few as a history but primarily to show that after all there is very little accomplished but by the slow evolution of thought added to thought until finally a principle of truth is established.

We must remember that at the time of our warehouse no water-cement ratio was known, and so field control of the strength was absolutely out of the question. It was entirely a matter of hit or miss.

The Georgia Power Company bought the warehouse in 1916 and has since used it for a general storehouse for many types of heavy electrical equipment. It has several times been thought to be overloaded and so a load survey of the structure was planned to ascertain just exactly how much live load might be allowed on the floors.

The survey was conducted by the author for the Power Company, using our most modern methods of design as given in the Report of the Joint Committee on Concrete and Reinforced Concrete, submitted August 14, 1924. A second survey had been given previously using the Building Code of Atlanta as the basis of survey but after testing a sample of the concrete to ascertain its strength, it was decided to use the former method since it allowed higher stress for higher strength concrete. Our concrete was some twenty-four years old and since it gains strength with age, we believe that this was a logical conclusion because in this way we take into account the additional strength gained by twenty-four years curing.

In order to check accurately the allowed live load, typical bays were measured carefully and the concrete broken into, and micrometer sizes taken of the steel reinforcement. An accurate set of levels was run on the building to obtain the average thickness of the slab and beams and to get the exact column lengths. At the close of field work, we went to the City Hall and obtained what little information was filed over there with the building inspector. This information was very enlightening. The building was drawn up as a slab, beam, girder column structure and was built as a slab, beam, column structure with columns smaller than called for on the drawing. In other words, the girders are completely left out.

This building shows many things about design that the engineering profession should know. The field man does not always follow close enough to the design as given in the drawings.

In our case the girders were entirely left out and because the building is "still standing" no one objects so very much. However, another case of carelessaness is shown in the fact that in one beam one-half of the steel reinforcement is missing. Perhaps, the construction foreman thought that he knew more than the designer or perhaps it was carelessness. At least, we see even in 1907 a very great need for inspection by the engineer of all structural works so as to eliminate any gross errors that might come up on the job.

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ATLANTA, GEORGIA.

JUNE, 1931. H. J. S.

CONTENTS

BOOK ONE

The History of Concrete

List of Illustrations

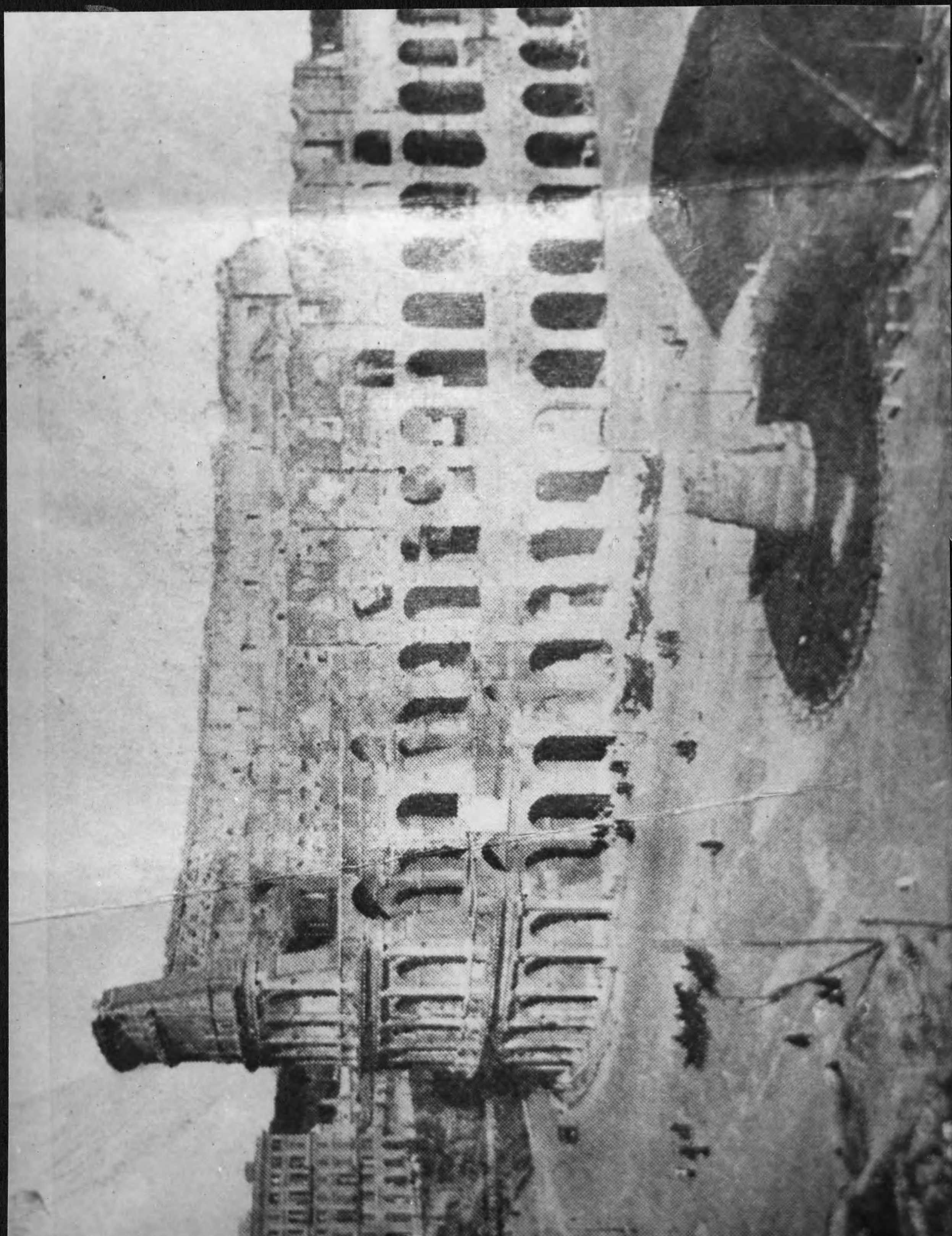

PLATE I

Reinforced Concrete

Reinforced Concrete! Steel and Cement! Man made structures as permanent as the eternal rocks! To the structural engineer these statements are synonomous. They picture lofty buildings, lasting roads, beautiful bridges and tremendous dams. What wonders of engineering are possible because of this construction giant!

The History of Concrete

No one knows when, where, or by whom concrete was invented or first employed. Its use as a material of construction dates back to the dim antiquity of prehistoric times, and examples of the ancient craft have come down to us with practically undiminished strength which have withstood the bombardment of nature's forces through all the centuries. Certain it is, however, that long before the dawn of authentic history, at a period back beyond the prying curiosity of scientific research, and while the earth was still young, people lived who knew a great deal about cement and its adaptability to the various needs of construction, as well as its wonderful powers of resistance to the disruptive action of natural forces.

The ancient Egyptians understood the use of hydraulic cement. It has been proven that in some of the marvelous constructions which endure as monuments of their engineering skill, they used a porous lava possessing hydraulic properties and containing the basic element necessary to the making of cement somewhat similar to the Portland cement of the present day. Many of the sarcophagi in which they placed their dead were made of artificial stone. The majestic pyramids, which for over 4,000 years have reared their stupendous forms above the Sahara and which still tranquilly laugh defiance at the ravages of time, were built in part of concrete. It is generally conceded that in the construction of their upper

PLATE II

tiers, at least, concrete was the material employed and the massive blocks of stone that have baffled past ages by the mystery of their transportation to such elevations were probably borne to their destination by the bucketful and formed directly in place. We have evidence that these blocks are of man's formation in the fact that breaks in some of them have revealed small pieces of wood embedded in the mass.

The Romans constructed many miles of highways, walls and aqueducts from hydraulic cements. We even find it used in the construction of some of their homes and temples which even today tell of the splendor that was once the mighty Roman Empire.

The word "Concrete" is itself of Latin origin, meaning "grown together" ("con", together; and "crescere", to grow) and implies a body formed by the condition of separate particles into a solid mass.

Roadbeds of concrete resounded to the thundering tread of the Roman legions as they went out to or returned from their conquests of the then known world. The famous Via Appia of which Caesar speaks so often in his Commentaries and over which the Apostle Paul entered Rome was underlaid with cement concrete and topped with paving stones. The latter have been worn away but the concrete is still intact just as when the Romans laid it. The aqueducts which supplied the Eternal City with water were built without reinforcement and are still in almost perfect condition. The cement lining of the Pont du Gard at Nismes, in Southern France, a Roman aqueduct built in the first century A.D. is still hard and smooth as when first put in place. The pools of King Solomon, nine miles from Jerusalem, were built of concrete and still furnish water for the City. Many residences of the Roman nobles were constructed of cement unfaced by brick or stone. Wood framing was used in casting the walls in much the same manner as wooden forms are used in concrete construction today. The Colosseum was built on piers and foundations of concrete while Middleton in his book "Ancient Rome", tells

PLATE III

us that the entire upper floor of the Atrium Vesta was formed of one great slab of concrete fourteen inches thick and having a span of twenty feet supported on edges with no intermediate supports. The Pantheon of Rome, a circular temple originally dedicated about the time of Christ, has a dome 142 feet in diameter supposed to be strengthened with iron rods and is still in nearly perfect condition after 1900 years of service.

A similar story as to the early use of concrete might also be written of the vanished races of the New World. The Peruvian builders in the days of the Incas employed concrete and some of their structures though built many centuries ago have endured to the present time. Even in our own North America, the use of this material antedates all historic records though it is of less enduring form. Twenty miles northeast of the City of Mexico are remains of a former but now vanished civilization in the shape of pyramids of masonry that were built partly of concrete. Ethmologists tell us that as far back as eleven thousand years ago, the remarkable race of men known as the Mound Builders living along what is now the Mississippi and Ohio Valley, were accustomed to boil salt water in kettles of artificial stone. Their pottery, specimens of which are the most enduring mementos of their advanced intelligence, contained clay and lime or sand which are used for concrete to the present day.

Concrete, we see, can therefore lay no claim to novelty for it is merely a return to principles once quite well known though not as perfectly known as today. The skill known to the ancients seems to have been a lost art to constructors of the Middle Ages. We find even in the elaborate cathedrals built in this period that the hydraulic cements have degenerated to a mere mix of silt and lime which crumble as the moisture evaporates. This necessitates a program of continuous repair to keep the beautiful structures erect.

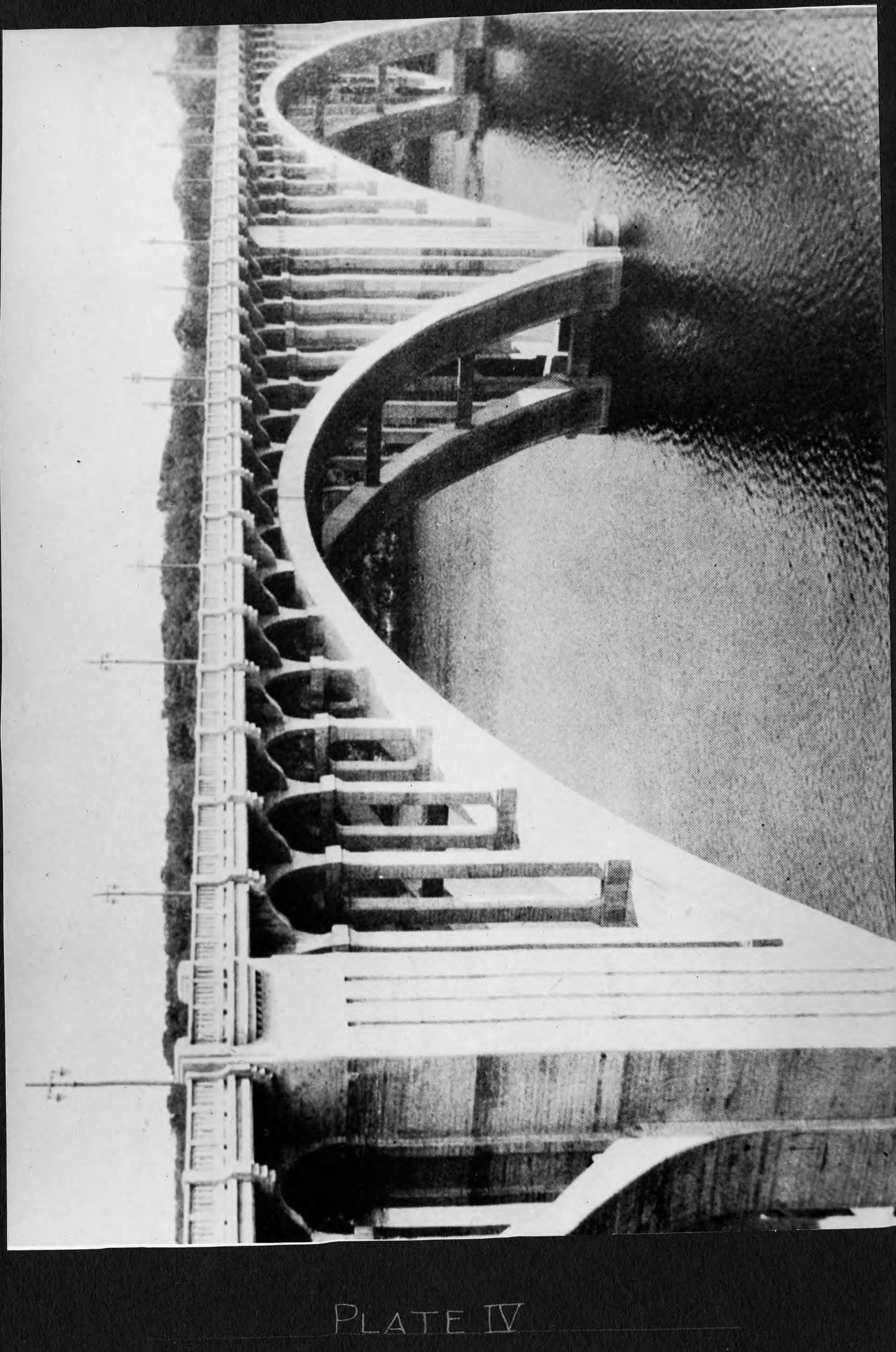

PLATE IV

About the beginning of the eighteenth century there came a revival in the demand for hydraulic mortars which was met by supplies of "Pozzuolana" from Italy, a mixture of hydrated lime and volcanic ash and "trass", a low grade natural cement, from Germany. The first true hydraulic cementing material, that is, one that hardens under water, was made in 1756 by the English engineer, John Smeaton, as a result of his searches for a proper binding material for building the third Eddystone Lighthouse. In 1796 we find the first natural cement and 1824 Joseph Aspdin, a mason of Leeds, England, patented Portland Cement, a mixture formed from burning slacked lime and clay - the crude forerunner of the present day material. The name Portland was chosen on account of the resemblance of the hardened cement to the building stone quarried on the Isle of Portland in the English Channel.

Even with the discovery of Portland Cement, the building world could find little use for the material until the correct proportioning of calcareous (consisting of lime) and orgillaceous (clayed) matter and the correct burning temperature necessary to insure a good product was found. It was in 1828 that a German chemist formulated the first theory of the action of the ingredients and their proper combining proportions to make a true Portland cement. Improvements came from time to time but progress was quite slow. Manufacture of Portland cement really started in Europe around the middle of the 19th Century, and the first Portland cement was brought to the United States in 1865. The first plant in this country to manufacture the product was started by David O. Saylor in 1872 who exhibited his material as a curiosity at the Centennial Exhibition in Philadelphia.

The manufacture of Portland cement lagged behind that of natural cement until the modern method of manufacture (burning the cement clinker in rotary kilns) was introduced in 1892, a process that

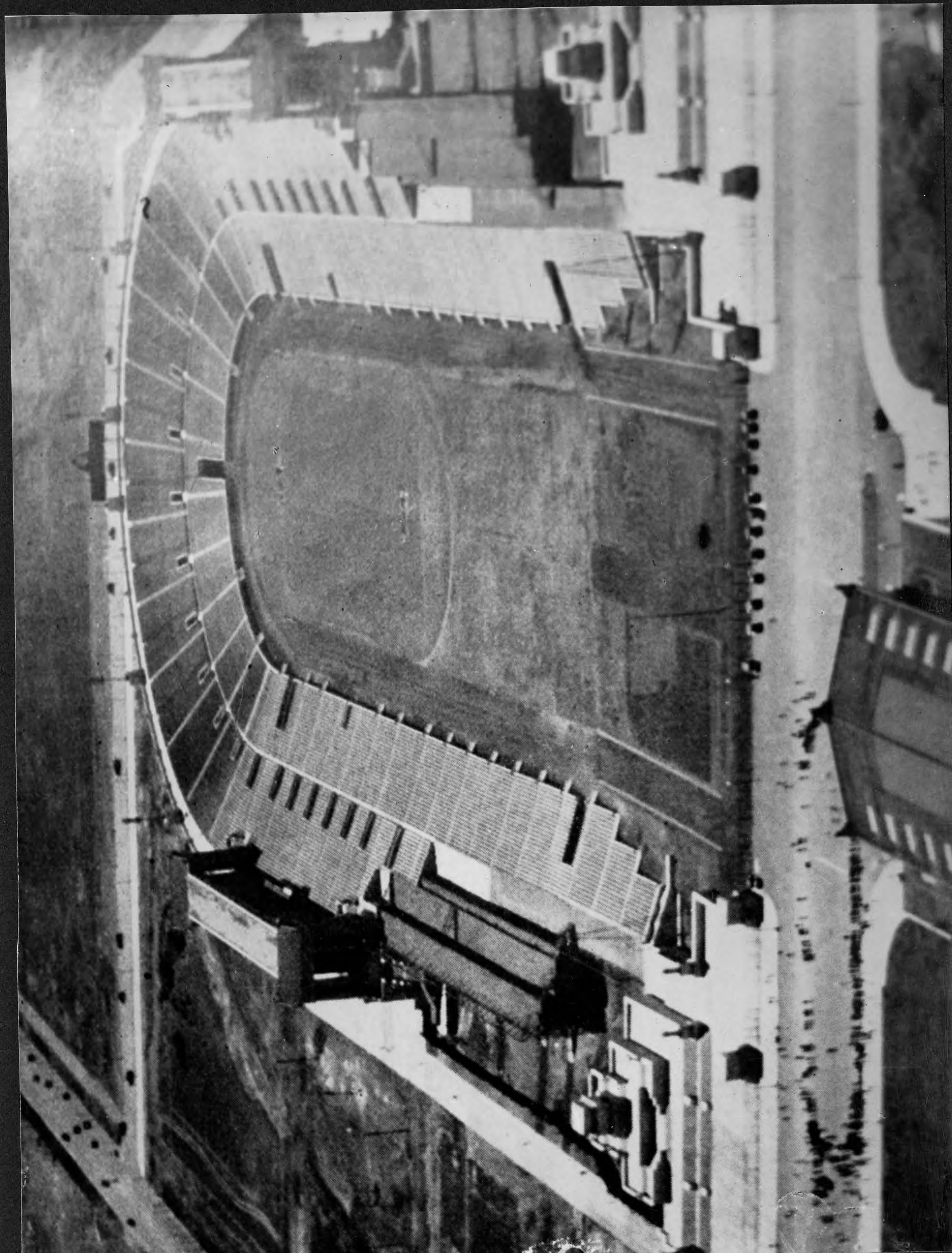

PLATE V

Thomas A. Edison spent countless hours helping to perfect. The rotary kiln is the largest piece of moving machinery used in all industry. Modern kilns as long as 375 feet, built in line with Edison's ideas, have at one end a hissing jet of flame 30 feet long and the cement ingredients travel toward this flame where they are burned at temperatures much hotter than those of a volcano - some 2500 to 3000 degrees Fahrenheit.

After the burning, the clinker is put through a grinding machine where it is pulverized so finely that four-fifths of the powder will pass through a sieve with 40,000 holes per square inch - a sieve woven finer than silk. With a little gypsum added to regulate its rate of hardening, this material, finer than flour, is now Portland cement - the cement that enables us to build not only with stone-like hardness but in any shape we desire - in fact a stone that we can mold.

After this invention, the production of Portland cement quickly mounted until now it ranks as one of the ten leading industries, an increase that tells eloquently of the increase in reinforced concrete construction.

In 1908 Bied in France and Spackman in the United States took out patents covering high-alumina cement that so far surpasses Portland cement in several important respects that its advent may mark an advance comparable to that made by the introduction of its older similar product. This is a high early strength cement due to its chemical composition - the use of a high-grade aluminum ore (Bauxite). It is not "quick setting". It affords nearly the usual time for mixing, transporting and pouring into forms but after setting its high strength develops with great rapidity. In fact, it will give the strength required of ordinary Portland cement in twenty-eight days at the end of twenty-four hours. This relieves concrete of one of its greatest drawbacks - the set up period required before load may be applied.

PLATE VI

One more phase of concrete making must be touched before we can say that a true history up to the present day has been presented. Man has long realized the uselessness of a finely made cement and a highly developed design (the latter we will cover in our next chapter) unless we can fulfill our assumptions of design in the actual structure, i.e., we must get the strength we design for in the field.

Much research has been conducted on determining a foolproof method for predetermining the strength of concrete used in modern structures. Duff A. Abrams, in charge of the Structural Materials Research Laboratory of Lewis Institute in Chicago, first advanced the now famous "Water-Cement Ratio Method" in 1918 urging that the strength of concrete of workable consistency is fixed by the amount of water used per bag of cement. His method and another the "Mortar-Void Method" by A. N. Talbot and F. E. Richart of the University of Illinois seem to be the two outstanding methods of proportioning at the present day.

The former method has been revised from the fineness-modulus method to a trial mix method by the Portland Cement Association under the supervision of F. R. McMillan, and is now the basis of design in structural concrete work.

In the future, the progress of scientific concrete knowledge is in the hands of the Portland Cement Association. This organization has contributed to the engineering and architetural world studies and researches of the greatest value. They follow the typical American engineering method of reasoning-deductive. A test is made in the laboratory for every study and conclusions drawn from the test after all curves representing the data are drawn up. Also to the research departments of our colleges and univer-

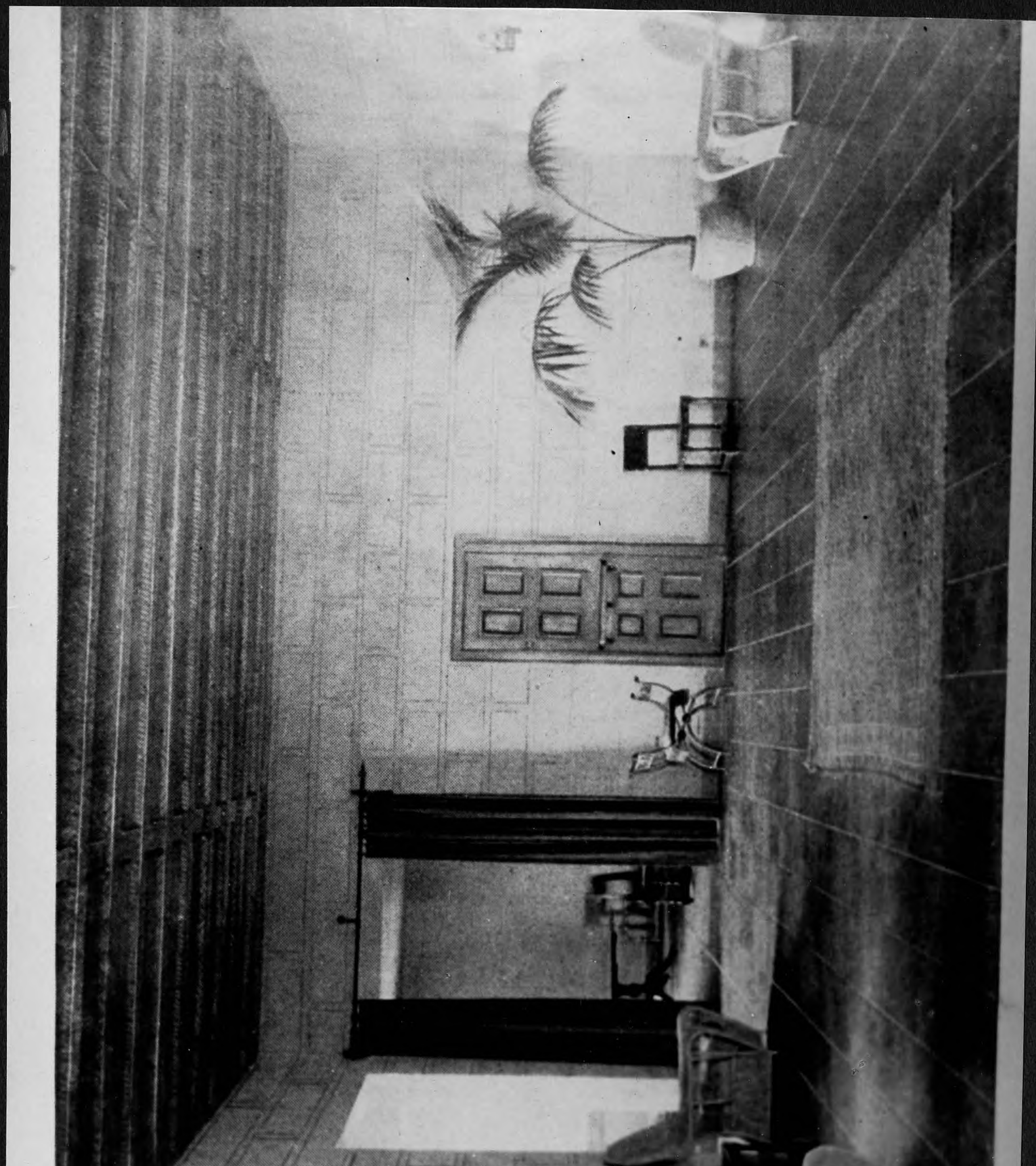

PLATE VII

sities do we owe much but to insure as bright a future as our past has been in the study of concrete making there must be a closer working together of our seats of learning and the practicing engineer. They must recognize their inter-dependence and utilize what the one can give the other. The advancement of science must never be hindered by any of the frailties of human emotions.

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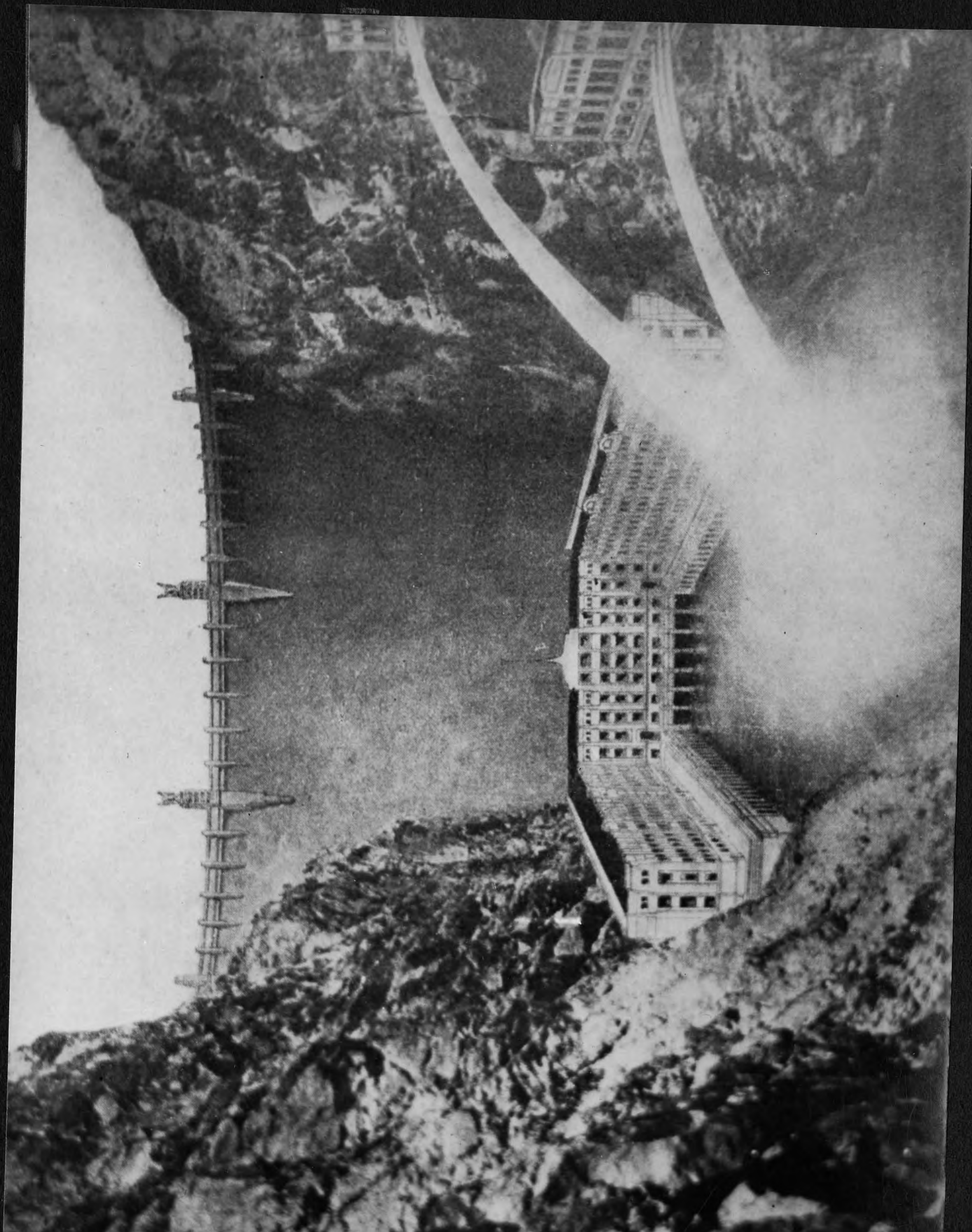


PLATE VIII

BOOK TWO

The Growth of Theories of Design of Reinforced Concrete

The growth of Theories of Design of Reinforced Concrete

The first constructors of works in reinforced concrete were not theorists. Several of them had not even an idea of the permanence or stability of the works they built. For that reason the theory of design followed a long time after the science of reinforced concrete construction had been quite well established.

The first reinforced concrete structure that we hear much of was a small boat built of that material by a Frenchman named Lambot in 1850. In 1854, an Englishman, Wilkinson, patented a true reinforced concrete floor slab and in 1861 Manier, a Parisian gardner , used metal reinforcement in garden tubes and pots. In the same year Francois Coignet published his statement of the principles of the new construction and sixteen years later Thaddeus Hyatt, an American, published a report on concrete beams combined with iron with several computations to determine the strength of them. However, comparatively little construction occurred until the German engineers, Wayss and Bauschinger investigated and reported on the Manier system in 1887. Wayss and Koenan made known a system of calculations which were later used in the design of the Manier slabs and arches. These formulae were empirical, however, and do not seem to impart the role of the concrete and steel in the combined material, reinforced concrete.

The studies undertaken on this subject have recognized from the beginning that the functions of members of reinforced concrete under load must depend on the elastic properties of the two materials. These properties, well-known for steel, were not so well-known for concrete but the theory was attempted and put into formulae in France in 1876. De Mazas applied these calculations to a structure of reinforced concrete. After him the problem was studied in the same country from 1894 to 1902 by Planat, Coignet, de Ledesco, Le Fort and Resal, while Neumann (1890), Spitzer, Mandl, Melan and Von Thullie (1896-97) followed with studies in Austria. After this came others, namely; Oestenfelt in Denmark, Lutken in Sweden, Sanders in The Netherlands and Ritter in Switzerland.

At the same time as these later theorists, the experimentalists worked on the question. However, they were a little late in establishing the law of deformation of concrete for it was not until the publication of Bach (1895-97) that sufficient data was obtained to base definite conclusions on.

For a long time the theorists groped about and in 1906 a standard text book of the day says, "While some of these theories are deduced from a few experiments, others are entirely theoretical and none are demonstrated to be absolutely true. This condition is due to the fact that not enough experiments have been made to finally establish any theory". *

The constructors would not work with the engineers in this problem. They preferred the empirical formulae far more for their designs. Mr. Hennebique established some of these formulae for girders and slabs of his system and his formulae quite closely resembled those in use today.

Certain constructors realized that pure empiricism must end if a general theory was to be developed and a method of calculation was developed which though not true in every sense of the word at least considered all the factors present.

Since this, engineers began to ask themselves if this combination did not impart to the combined materials new properties quite unlike the properties of the two separated and so in 1899 Considere published a widely studied treatise on the subject. At about the same time Harel de La Noe also studied and wrote quite a little on the theory of reinforced concrete.

* Reinforced Concrete - Buell and Hill - 1906 - P.17

In this country, the pioneer was W. E. Ward who built a reinforced concrete house in Port Chester, New York, in 1872. His primary object was to make a fireproof building and his system of heating was to place concrete double walls in each room so that heat might be conveyed to the rooms from a furnace set in the basement.

The early development of the new system took place in the United States in California. In 1877, H. P. Jackson applied Mr. Hyatt's invention to building construction, using iron blades set horizontally on edge with round iron cross wires for the reinforcement. In 1884 and 1885 E. L. Ransome built a warehouse; a few years later a factory building; in 1888 and 1889 the building housing the California Academy of Science and the Museum of Leland Stanford Junior University in 1892. Besides the above mentioned designers, Edwin Thacker and F. Von Emperger also did a great deal to introduce this system of construction, especially along bridge building work.

The Engineering News of July 30, 1903, has an interesting article on the daring design and construction of a sixteen story skyscraper, the Ingalls building of Cincinnati finished in the early part of 1904. It was built using the Ransome system, using a system of beams and girders not quite unlike our standard systems today.

All the concrete buildings up to this time were modeled after the steel and timber structures of the day. As a result the monolithic character of the concrete was ignored almost altogether and all possible economy was therefore not affected. As the principles of the material became better known engineers began to depart from the beaten paths of practice in other types and to treat it as having entirely different character from the then prevailing types, wood and steel.

The next step was to eliminate the many beams and girders and thereby cut down the cost of formwork and placement of steel. This was accomplished by Mr. C. A. P. Turner of Minneapolis, who invented the famous "Mushroom" floor in 1906.

To trace the history further is hardly needed. Pictures will suffice to show the advances of constructions in reinforced concrete. We feel that this method will better convey the idea of what has been done than to take further time for discussion because from 1905 on the structures of this material are quite profuse. We can only attempt to trace the stories of the most outstanding.

However, we feel that a short resume of the various theories of design will be of interest and with this idea in mind we have considered the most outstanding and have finished with a detail of the final Joint Committee Report on the subject.

From the history, we can well see that at the time of building our warehouse the theory of reinforced concrete was not well established and that we are to expect anything almost in our load survey.

The following is a short resume of the theories of design as they developed during the groping "Dark Ages" of structural design.

A Resume of the Various Theories of Reinforced Concrete Design*

Mazas-Neumann Method - The highway engineer Mr. de Mazas in France and Prof. Neumann in Austria seem to have been the first to apply the theory of elasticity to reinforced concrete. The method they have used was taken up and developed later by different writers, especially Lefort and Resal, chief highway engineers (France) and by Mandi, captain in the engineering corps (Austria).

This method supposes that the stress in the beam varies as a straight line A" O B" (Fig. 2) and implies the following hypotheses:
The coefficient of elasticity of concrete for tension is the same as for compression and it remains the same under the usual load limits.

* Translated and rewritten in part from the French Treatise "Beton Arme" - Paul Christophe, Page 494 to 502.

VARIOUS THERIES OF STRESS VARIATION

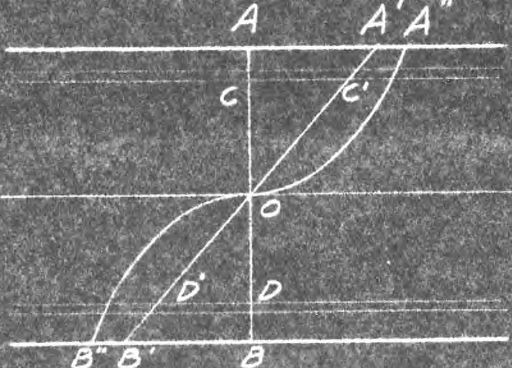


FIG. 1

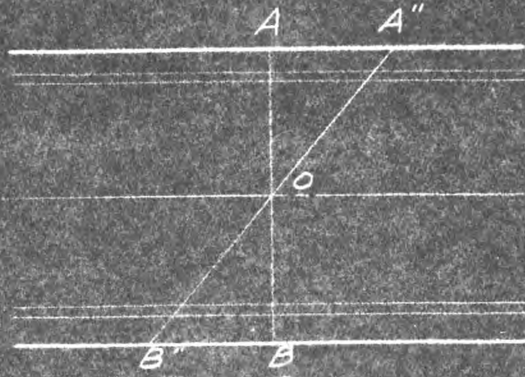
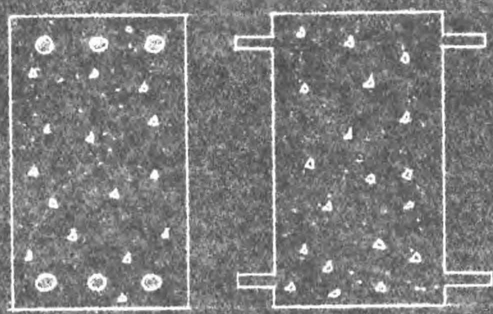


FIG. 2


FIGS. 3 & 4

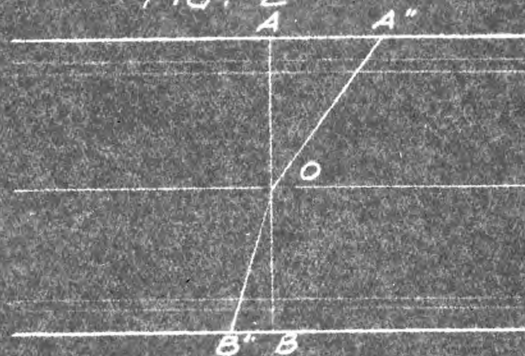


FIG. 5

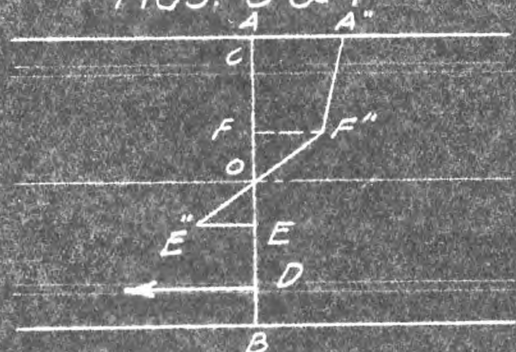


FIG. 6

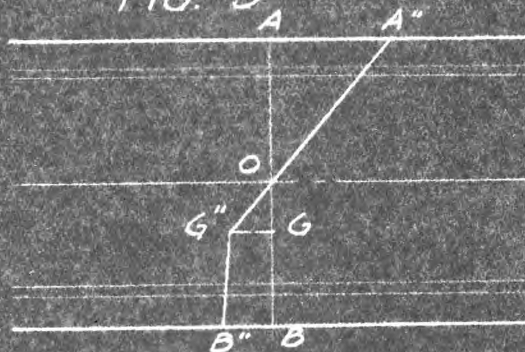


FIG. 7

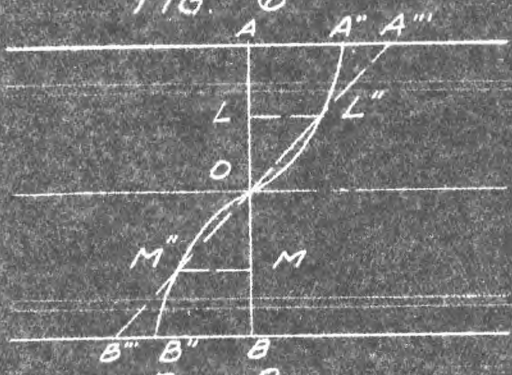


FIG. 8

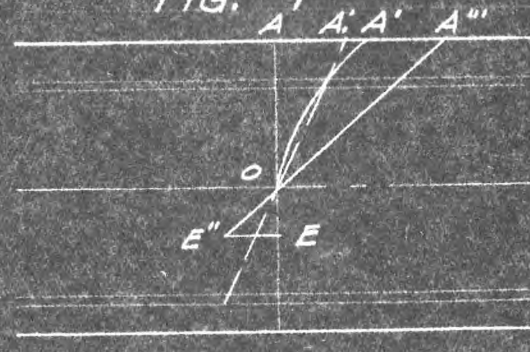


FIG. 9

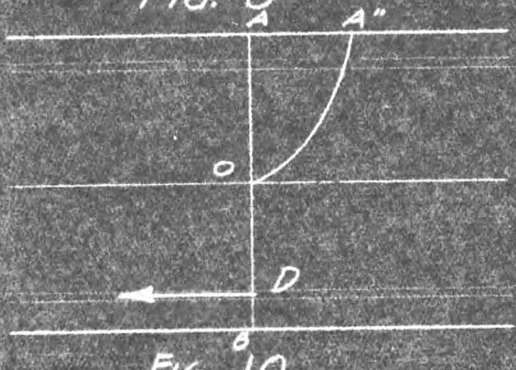


FIG. 10

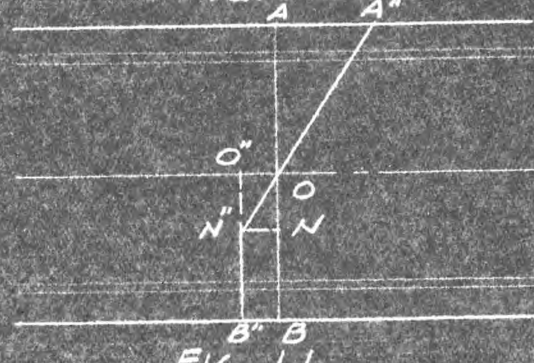


FIG. 11

This method cannot be refused the merit of being simple, but the concrete being assimilated with the iron and with very uniform elastic properties (we question today - 1931) it is easy to approach the study of a piece of reinforced concrete as a homogeneous beam. Since the steel is tightly bonded in the concrete, it is sufficient to multiply the stress at the surface of each piece of steel by E_s/E_c = n to get the stress in equivalent concrete. The new section thus obtained treated with the usual formulae for the bending of homogeneous beams gives directly the change of stress of the concrete under compression and tension. As for the change of stress in the reinforcement, it is equal to that of the concrete calculated at the same level and multiplied by n.

Unfortunately this theory in no way fits real conditions. To get an idea of this it suffices to refer one to our present assumptions given in the Joint Committee Report.

When the piece of reinforced concrete carries only a relatively light load it is doubtless possible to consider the coefficient of elasticity of the concrete as constant both under tension and compression. However, since the resistance of the concrete in tension is a great deal less than it is in compression, the concrete fibres situated at the lower or tension face of the beam are rapidly stressed beyond their elastic limit. The time comes then when the line OB" is not a straight line even though OA" may approximate one and the neutral axis will move up toward the upper or compression face. Failure takes place on the tension side when the tension strength of the concrete is surpassed and the reinforcement then takes up the total tension stress.

There results from the preceding that the hypotheses of de Mazas and Neumann can only explain with some chance of exactness the conditions at the very beginning of loading without even showing the use of the reinforcement. This method would fail slightly after the load had passed the dead load of the beam itself.

Melan Method - Prof. Melan wished to correct the inexactness of the above method by stating that there exists an inequality between the coefficients of elasticity of concrete in extension and compression but he continues to suppose that each one of these coefficients has a constant value.

The stress then varies as shown in A"OB" (Fig. 5) composed of two straight lines of different slope.

Again we have a method using the tension in the concrete in computations - an hypothesis that has been antidated quite a while ago in concrete design practice.

Melan did not use his theory in his arch design but proposed to use the system in beam practice.

Coignet and de Tedesco Method - The majority of workers in reinforced concrete in the last few years of the nineteenth century state as a point of departure from the empirical formulae which they use that one should not pay attention to the tension strength of concrete.

Edmond Coignet and de Tedesco, civil engineers of Paris, were the first to hunt for a theory which, taking this supposition as a basis, should pay attention to the elastic properties of the materials under consideration.

Like the preceding writers, they adopted for the coefficient of elasticity in compression a constant value. The variation OA" of the compression stress is still therefore a straight line. In the tension zone OB" the concrete did not enter any longer. Only the tension of the reinforcement is taken into consideration.

The theory of Coignet and de Tedesco was unfortunately flase because of errors in calculation. They supposed that the center of application of the pressure resulting on OA was found in the middle of this height while its true position is at 2/3 of OA.

Von Thullie Method - Prof. Von Thullie, in the series of studies which he published on the calculation of reinforced concrete constructions, was the first to give a complete exposition of the mode of resistance of the heterogeneous material.

Von Thullie considers that a mass of reinforced concrete bent by an exterior load passes successively through two distinct phases; first, during which the concrete works both under extension and under compression, and the second which begins by the breaking of the concrete in the zone under pressure.

For the first phase Von Thullie adopts the hypotheses of Neumann (Fig. 2).

In the second, he began by admitting as do Coignet and de Tedesco that the law OA" of the pressures remains rectilinear. Later he wished to treat the question more exactly, and considered the spot OA" as being formed by two straight lines OF" and F"A" (Fig. 6). The first of these straight lines forms the prolongation of the law of extensions which extends as far as the point E so that the tension EE" represents the resistance limit of the concrete to the extension (Under extension). However, as the tensions of the triangle OEE" in relation to the neutral fibre O only give a very slight moment, the latter is negligible in the calculation.

Von Thullie gives therefore for the coefficient of elasticity of the concrete two successive values. The first applicable to the extension as well as to the compression, is introduced into the calculation of the first phase and into that of the second as far as the limit FF".

Beyond that limit which stress limit is about 710#/Sq. In. it takes a lesser value. (Von Thullie admits as the first value that $E_c =$ 2,850,000 #/Sq. In. whence n = 10 and for the second he considers $E_c =$ 1,425,000 #/Sq. In. whence n = 20.

According to Von Thullie the resistance of bent pieces of reinforced concrete should be determined in the view of two limits of security relating respectively to each one of the two phases of the work.

It is convenient first of all to assure oneself that the concrete will not be cracked in the part extended. By the aid of formulae relatively to the first phase (Fig. 2), one must verify whether the extension rate BB" does not surpass the resistance to extension. Von Thullie is of the opinion however that limitation of fatigue should not be exaggerated and he admits extension forces going as high as 285#/Sq.In., a figure that may be reduced, he says, to 215 #/Sq.In. to avoid all danger of breaking.

M. Von Thullie advises not to be contented with this verification and to assure oneself besides whether the concrete might break, that is to say, in the hypotheses of the second phase, whether the concrete (under compression) and the iron (under extension) would attain their breaking limit only under a load a great deal greater than the one the piece should support.

Von Thullie's method although much to be preferred to the preceding ones still is open to several objections.

He considers in the first phase allowable stresses which are very close to the breaking point. Now it is more than probable that the elasticity of the concrete is already altered under such great pressures. It is therefore difficult to admit that in these conditions the law OB" of the extensions (Fig. 1) can be represented by a straight line.

Besides, one cannot imagine what can be the use of the relative calculation in the first phase. It is certainly desirable to avoid the production of cracks in the concrete, but the condition of which it is a question cannot be assured with the allowable stresses indicated by Von Thullie. The security they would give, even while reducing the extension rates, would only be deceiving, for the concrete can be cracked before the loading.

The condition relative to the second phase offers much greater importance. It alone permits, with his hypotheses, the explanation of the important part played by the reinforcement in resistance. If in fact, in the first phase, one places the tension of the concrete less than 285#/Sq.In. one proves, by admitting the value of 10 for n that the steel develops at most a stress of 2850 #/Sq. In. So long as the concrete is not broken, according to the theory of Von Thullie, there would only be gotten a very insignificant benefit from the presence of the reinforcement. The latter really enters into the computation only in the second phase.

Von Thullie himself admits besides that it is the conditions of this phase which ought to serve to determine the section of reinforcement. But to make this calculation, he requires again a state very close to the breaking point. He considers a stress occurring of 1780 to 2850 #/Sq. In. for the concrete under compression and of 49,800 #/Sq.In. for the steel under tension.

Now, if one acts thus, the compression curve OA" is doubtless not straight, which justifies the second hypothesis of Von Thullie (Fig. 6), but neither is the deformation of the metal submitted to the law of proportions since the limit of elasticity of the metal is surpassed.

The best thing then, it seems, is for the calculation of the second phase to be made in the usual allowed stresses, both for the concrete and the metal. By acting thus, one conforms to the usage adopted for the calculation of homogeneous pieces. He admits, basing on experiments of Hartig that the coefficient of elasticity of the concrete is constant up to a stress of 710 #/Sq. In. The working stress of the concrete being generally below this figure, one can by the hypotheses of Von Thullie, replace the line E"F"A" (Fig. 16) by a straight line. The hypotheses thus defined are, in our opinion, the most rational.

He, himself, has noticed besides that the complication of the formulae given by his second hypothesis (Fig. 6) is hardly justified, the results not being greatly different from those of the one which he admitted first.

Ostenfeld Method - The method of calculation proposed by Prof. Ostenfeld is absolutely analogous to the preceding one. Like Von Thullie he has two phases.

As we have remarked Von Thullie gets away from real conditions when he holds as rectilinear, in the first phase (fig. 2), the method of deformation of OB" while the stress BB" is close to breaking. Ostenfeld wished to correct this error. Basing on the results of several experiments made by Grut and Nielsen he admits that the law of extensions can be represented by two straight lines such as OG" and G"B"(Fig. 7). The coefficient of elasticity relative to the part GO of the section so that GG" equals 114 #/Sq." is equal to the coefficient in compression which is supposed constant. It then diminishes suddenly and retains a constant value on the stress side BB" of 199 or 228#/Sq.In. Ostenfeld adopts a coefficient of elasticity of 3,550,000 and 995,000 #/Sq.In. which give values of n equal to 8 and 29.

For the second phase, Ostenfeld adopts the first of Von Thullie's hypotheses. Ostenfeld's method would have some chance of being preferred to Von Thullie's if it were useful to consider concrete as working under tension, which we deny. But this hypothesis has not sufficient basis on experiment.

Sanders Method - Other authors have sought to perfect the calculation of reinforced concrete by adopting an irreproachable definition of the curve A"OB" (Fig. 1) of elastic tensions. Sanders, engineer of the Dutch firm "Amsterdamsche fabriek van cement-i jzerwerken", has gone as far as possible in this direction.

According to the researches of Bach on the elasticity of concrete under compression, the law deformation as a function of the stress may be expressed by the following formula.

$$e = \frac{1}{E_c} p^x$$

where

e = unit deformation

E_c = initial modulus of elasticity

p = unit stress

x = a power varying (according to experiments of Bach and Schule) from 1.10 to 1.16 for various mixes of cement.

Sanders adopts this formula for the equation of the curve OA" (Fig. 8).

As for the law of deformation in tension he says that it is of similar form and he defines the curve OB" by a similar formula but adopts different values for E_c and x.

He studied the tension and considered it in his theory. He thought that the members should be so designed that they will not fail under tension.

For the reasons given for the Von Thullie method (first phase), first we ought to point out as being very irrational this condition which does not permit any explanation of the part played by the reinforcement.

The extraordinary complication of the formulae to which sanders arrives is not justified by any real necessity for his taking of Bach's equation for stress-strain variations is not altogether true, since other experimenters have gotten other equations for this relation.

Nothing hinders one from supposing that the compressed part OA of the section AB instead of remaining straight really takes a curved form, in fact, we are nearly right when we say that more concrete does this in bending. Bach's law can be used only in the limits of usual tension because the extension at B could not pass a law working stress without causing danger of failure.

In view of practical applications, Sanders has given a simplified theory by replacing the curves OA" and OB" by the straight lines OA"' and OB"' (Fig. 8), which cut them at the height of the points L and M situated at two-thirds of the heights of the compressed and extended zone. This is nothing more than Melan's method.

Spitzer-Lutken Method - In the same order as the preceding method, Spitzer, chief engineer of the firm G. A. Wayss of Vienna, and Prof. Lutken have proposed to consider the curves OA" and OB" as parabolas of the second degree.

These theories are open to the same criticisms as the Sanders method for they do not even translate faithfully a deformation law verified by practice. In Bach's formula the coefficient "x" is no closer to 1 than to 2. The law is closer then to a straight line than to a parabola when the stresses remain within ordinary limits. The parabola seems better to suit the law of variation of tensions.

Ritter Method - Prof. Ritter has likewise proposed to assimilate the law of elastic tension of concrete to a parabola, but he makes this hypothesis only for compression and neglects the extension forces of the concrete below the neutral fibre.

Outside of these hypotheses which he considers especially in the case where the load is close to breaking, Ritter has developed another method simpler, and one which he considers as sufficient in practice.

This method is based on the hypotheses of Mazas and Neumann, for by beginning with these hypotheses Ritter determines the position of the neutral fibre. However, considering that the concrete cannot bear in extension the considerable force that practical use demands, he calculates the reinforcement by imagining the concrete to be cracked, but he does not modify the position of the neutral fibre. However, we know that when the tension side has failed the neutral axis goes toward the compression face and we then increase the lever arm of the reinforcement.

Under these conditions, Ritter remarks, it is as valuable to suppose that the neutral fibre is invariably situated half way up the piece. If one adopts this hypothesis, one falls into the empirical method of Koenen and we are not dealing with empiricism as it is beyond usage in general design.

Considere's Method - M. Considere, chief engineer in France in 1902 points out that concrete is far stronger when reinforced than plain. He says that the concrete will resist the same stress as the steel up to the elastic limit of the steel. The fissures appearing in the tension face are therefore immaterial as long as the elastic limit of the steel is not passed.

When reinforced concrete deflects under increasing load the concrete of the tension face is subject to stress which increases rapidly up to the elastic limit. From this point on, the concrete forced by the steel to all the elongation of its fibres, its particular molecular arrangement causes stretching more and more without a corresponding increase in tension. He says that the stretched concrete continues thus to provide a resisting moment which is added to that of the reinforcing steel. Also, he says that the elongation imposed on the concrete is of no great value but as regards its flexural resisting moment it is important because this keeps the compressed concrete and reinforcing from reaching their breaking points and elastic limits respectively as soon.

Based on these theories, M. Considere has established different methods of computation more or less complete. We will look at the one that he says is quite adequate for the needs of practice.

The reinforced concrete piece is considered as being in the second phase of resistance, that is to say, it is in the period of tensile ductility, at the period when the elastic limit is being reached in the steel.

The compression side in flexure is taken as a straight line (Fig. 11) and the tension side is formed by two straight lines - the one ON", a prolongation of OA", the other N"B" parallel to OB, i.e., the tension in the concrete is constant along NB. He therefore calculated the stress caused by load when the value of the allowed tension and compression are set down.

From the above methods we see how many different ideas were given out in the growth of a rational theory of design. These are those methods based on a theory alone; the empirical methods were legion. From several text books of the day of our warehouse we see that many "rules of thumb" and empirical methods were recommended. Bond was only mentioned then, while today many structures could be designed on bond considerations alone.

To really show what was finally the outcome of these many thoughts the design portion of the 1924 Report of the Joint Committee on Concrete and Reinforced Concrete was added as a closing to Book II.

The engineering profession realized the need of an authoratative report on the theories of reinforced concrete design, so in 1904 a committee was organized to consider and report on the best methods. A final report was presented in 1916 and again in 1924, the last mentioned being the one used today. This final report of 1924 is a very fine piece of workmanship. It will suffice until we advance still further in a few of the details because in the main it will not be changed.

The following is the present theory of design as given in the Joint Committee Report.

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CHAPTER IV

Design

The formulas given in this chapter are in conformity with the report of the Joint Committee on Concrete and Reinforced Concrete and are accepted by the Industry as standard. The moment, shear, bond, flat slab and column sections are in exact conformity with "Tentative Building Regulations for Reinforced Concrete" developed as explained on the previous page. They are included for the information of the designer.

Design Assumptions

The design of reinforced concrete members under these specifications shall be based on the following assumptions:

(a) Calculations are made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending, shearing distortions being neglected.

(c) The modulus of elasticity of concrete in compression is constant within the limits of working stresses and the distribution of compressive stress in beams is rectilinear.

(d) The modulus of elasticity of concrete in computations for the position of the neutral axis, for the resisting moment of beams, and for compression of concrete in columns, is as follows:

$$E_c = 1000f'_c$$

That is:

$$n = \frac{E_s}{1000f'_c}, \quad \text{or}$$

$$n = \frac{30{,}000}{f'_c}$$

(e) In calculating the moment of resistance of reinforced concrete beams and slabs the tensile resistance of the concrete is neglected.

(f) The bond between the concrete and the metal reinforcement remains unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion to their moduli of elasticity.

(g) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced concrete columns.

Flexure of Rectangular Reinforced Concrete —Beams and Slabs

(1) Reinforced for Tension Only

Symbols

A_s = effective cross-sectional area of metal reinforcement in tension in beams.

b = width of rectangular beam.

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement.

E_c = modulus of elasticity of concrete.

E_s = modulus of elasticity of steel.

f_c = compressive unit stress in extreme fiber of concrete.

f_s = tensile unit stress in longitudinal reinforcement.

j = ratio of lever arm of resisting couple to depth d.

k = ratio of depth of neutral axis to depth d.

M = bending moment or moment of resistance in general.

n = E_s/E_c — ratio of modulus of elasticity of steel to that of concrete.

p = ratio of effective area of tension reinforcement to effective area of concrete in beams $= A_s/bd$.

z = depth from compression surface of beam or slab of resultant of compressive stresses.

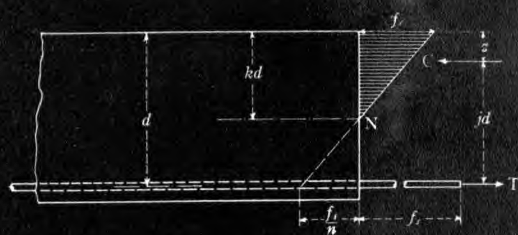


Fig. 1.—Nomenclature for Concrete Beam Reinforced for Tension.

Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn \quad \ldots\ldots\ldots\ldots (1)$$

Arm of resisting couple,

$$j = 1 - \frac{k}{3} \quad \ldots\ldots\ldots\ldots (2)$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \ldots\ldots\ldots\ldots (3)$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2} \quad \ldots\ldots\ldots\ldots (4)$$

Steel ratio for balanced reinforcement,

$$p = \frac{1}{2} \cdot \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)} \quad \ldots\ldots\ldots\ldots (5)$$

(2) Reinforced for Both Tension and Compression

Symbols

d' = depth from compression surface of beam or slab to center of compression reinforcement.

f'_s = compressive unit stress in longitudinal reinforcement.

p' = ratio of effective area of compression reinforcement to effective area of concrete in beams.

All other symbols as defined in (1).

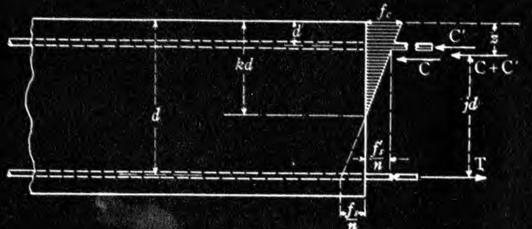


Fig. 2.—Nomenclature for Concrete Beam Reinforced for Tension and Compression.

Position of neutral axis,

$$k=\sqrt{2n\left(p+p'\frac{d'}{d}\right)+n^2(p+p')^2}-n(p+p') \quad \text{..(6)}$$

Position of resultant compression,

$$z=\frac{\frac{1}{3}k^3d+2p'nd'\left(k-\frac{d'}{d}\right)}{k^2+2p'n\left(k-\frac{d'}{d}\right)} \quad \text{..........(7)}$$

Arm of resisting couple,

$$jd=d-z \quad \text{..................(8)}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c=\frac{6M}{bd^2\left[3k-k^2+\frac{6p'n}{k}\left(k-\frac{d'}{d}\right)\left(1-\frac{d'}{d}\right)\right]} \quad \text{...(9)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s=\frac{M}{pjbd^2}=nf_c\frac{1-k}{k} \quad \text{............(10)}$$

Compressive unit stress in longitudinal reinforcement,

$$f'_s=nf_c\frac{k-\frac{d'}{d}}{k} \quad \text{...............(11)}$$

(3) Reinforced Concrete T-Beams

Symbols

b = width of flange of T-beam.

b' = width of stem of T-beam.

t = thickness of flange of T-beam.

All other symbols as defined in (1) and (2).

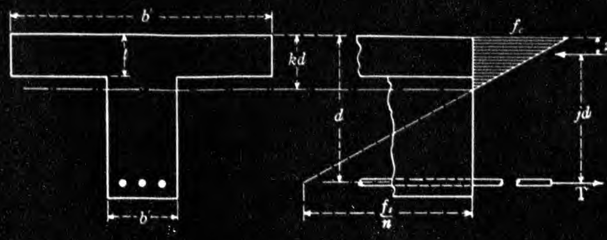


Fig. 3.—Nomenclature for Reinforced Concrete T-Beam.

Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(a) Neutral Axis in the Flange.

Use formulas for rectangular beams and slabs.

(b) Neutral Axis below the Flange.

Position of neutral axis,

$$kd=\frac{2ndA_s+bt^2}{2nA_s+2bt} \quad \text{.............(12)}$$

Position of resultant compression,

$$z=\left(\frac{3kd-2t}{2kd-t}\right)\frac{t}{3} \quad \text{............(13)}$$

Arm of resisting couple,

$$jd=d-z \quad \text{................(14)}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c=\frac{Mkd}{bt(kd-\frac{1}{2}t)jd}=\frac{f_s}{n}\left(\frac{k}{1-k}\right) \quad \text{......(15)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s=\frac{M}{A_sjd} \quad \text{...............(16)}$$

Formulas 12, 13, 14, 15 and 16, neglect compression in the stem. The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem;

Position of neutral axis,

$$kd=\sqrt{\frac{2nd\ A_s+(b-b')t^2}{b'}+\left(\frac{n\ A_s+(b-b')t}{b'}\right)^2}-\frac{n\ A_s+(b-b')t}{b'} \quad \text{.........(12a)}$$

Position of resultant compression,

$$z=\frac{(kdt^2-\frac{2}{3}t^3)b+[(kd-t)^2(t+\frac{1}{3}(kd-t))]b'}{t(2kd-t)b+(kd-t)^2b'} \quad \text{.....(13a)}$$

Arm of resisting couple,

$$jd=d-z \quad \text{...................(14a)}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c=\frac{2Mkd}{[(2kd-t)bt+(kd-t)^2b']jd} \quad \text{..........(15a)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s=\frac{M}{A_sjd} \quad \text{...................(16a)}$$

In T-beam construction the slab should be built integral with the beam. The effective flange width to be used in the design of symmetrical T-beams should not exceed one-fourth (¼) of the span length of the beam, and its overhanging width on either side of the web should not exceed eight (8) times the thickness of the slab nor one-half (½) the clear distance to the next beam.

For beams having a flange on one side only, the effective overhanging flange width should not exceed one-twelfth (1/12) of the span length of the beam, nor six (6) times the thickness of the slab, nor one-half (½) the clear distance to the next beam.

Where the principal reinforcement in a slab which is considered as the flange of a T-beam is parallel to the beam, transverse reinforcement should be provided in the top of the slab. This reinforcement should be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars should not exceed five (5) times the thickness of the flange, or in any case eighteen inches (18″).

Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions relating to the spacing of bars and relating to the placing of concrete shall be fully met. In no case shall the area of steel in compression at any cross-section adjacent to the support exceed two per cent (2%) of the cross-sectional area of the stem of the beam in that section.

The overhanging portion of the flange of the beam should not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

Isolated beams in which the T-form is used only for purpose of providing additional compression area, should have a flange thickness not less than one-half (½) the width of the web and a total flange width not more than four (4) times the web thickness.

(4) Lengths and Moment Formulas*

Symbols

l = span length of beam or slab (generally distance from center to center of supports—see following paragraph);

w = uniformly distributed load per unit of length of beam or slab;

I = moment of inertia of section about the neutral axis for bending;

All other symbols as defined in (1), (2), and (3).

The span length (l) of freely supported beams and slabs should be the clear span plus the depth of beam or slab but should not exceed the distance between centers of the supports.

The span length for continuous or restrained beams built to act integrally with supports should be the clear distance between faces of supports.

For continuous or restrained beams having brackets built to act integrally with both beam and support and of a width not less than the width of the beam and making an angle of forty-five degrees (45°) or more with the horizontal, the span should be measured from the section where the combined depth of the beam and bracket is at least one-third (⅓) more than the depth of the beam. No portion of such a bracket should be considered as adding to the effective depth of the beam.

Brackets making an angle of less than forty-five degrees (45°) with the horizontal may be considered as increasing the effective depth of the beam, but not as decreasing the span length.

Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

The depth of beam or slab should be taken as the distance from the centroid of tensile reinforcement to the surface of the structural slab. Any floor finish not placed monolithic with the floor should not be included as a part of the structural member. When the finish is placed monolithic with the structural slab, if the use of the finished floor is such that unusual wear would result, an additional depth of one-half inch (½″) over that required structurally should be provided.

For the purposes of calculation, the point of inflection in beams and slabs of equal spans symmetrically loaded should be assumed to be located at the fifth point of the span.

The clear distance between lateral supports of a beam should not exceed thirty-two (32) times the least width of compression flange.*

(A) Freely supported or slightly restrained continuous beams or slabs of approximately equal span; uniform load.

Beams and slabs of approximately equal spans freely supported or built to act integrally with beams, girders or other slightly restraining supports, or beams and slabs built into brick or masonry walls in a manner which develops only partial end restraint, and carrying uniformly distributed loads should be designed for the following moments at critical sections:

(a) Beams and slabs of one span,
Maximum positive moment near center,

$$M = \frac{wl^2}{8} \quad \text{.................} (17)$$

(b) Beams and slabs continuous for two (2) spans only,
Maximum positive moment near center,

$$M = \frac{wl^2}{10} \quad \text{.................} (18)$$

Negative moment over interior support,

$$M = \frac{wl^2}{8} \quad \text{.................} (19)$$

(c) Beams and slabs continuous for more than two (2) spans,
Maximum positive moment near center and negative moment at support of interior spans,

$$M = \frac{wl^2}{12} \quad \text{.................} (20)$$

*The subject matter from Section (4) to the end of this chapter is in accord with good engineering practice, conforms exactly to "Standard Regulations for Reinforced Concrete" recommended by Committee 501 of the American Concrete Institute and the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute and is generally applicable. Nevertheless, the reader is cautioned to investigate carefully the particular building code under which he is working for possible differences, before proceeding as outlined in these sections.

*No variations from this rule should be made, except under the most unusual circumstances, and then only when the unit stress to be used in the design is determined by the following formula:

$$f_c = \frac{f_c'}{53}(53 - l)$$

Maximum positive moment near centers of end spans and negative moment at first interior support,

$$M=\frac{wl^2}{10}\quad\ldots\ldots\ldots\ldots\ldots\ldots(21)$$

(d) Negative moment at end supports for cases (a), (b), (c) of this section,

$$M=\frac{wl^2}{24}\quad\ldots\ldots\ldots\ldots\ldots\ldots(22)$$

(B) Fully restrained continuous beams or slabs of approximately equal span; uniform load.

Beams and slabs of approximately equal span built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall [except when they fall into class (A)] be designed for the following moments at critical sections:

(a) Interior spans,

Negative moment at interior supports except the first,

$$M=\frac{wl^2}{12}\quad\ldots\ldots\ldots\ldots\ldots\ldots(23)$$

Maximum positive moment near centers of interior spans,

$$M=\frac{wl^2}{16}\quad\ldots\ldots\ldots\ldots\ldots\ldots(24)$$

(b) End spans of continuous beams or slabs and beams of one span in which I/l is less than twice the sum of the values I/h for the exterior columns above and below, at one end, which are built into the beam:

Maximum positive moment near center of span and negative moment at first interior supports,

$$M=\frac{wl^2}{12}\quad\ldots\ldots\ldots\ldots\ldots\ldots(25)$$

Negative moment at exterior supports,

$$M=\frac{wl^2}{12}\quad\ldots\ldots\ldots\ldots\ldots\ldots(26)$$

(c) End spans of continuous beams, and beams of one span, in which I/l is equal to or greater than twice the sum of the values of I/h for the exterior columns and below, at one end, which are built into the beam:

Maximum positive moment near center of span and negative moment at first interior support,

$$M=\frac{wl^2}{10}\quad\ldots\ldots\ldots\ldots\ldots\ldots(27)$$

Negative moment at exterior support,

$$M=\frac{wl^2}{16}\quad\ldots\ldots\ldots\ldots\ldots\ldots(28)$$

In (b) and (c) "I" represents the moment of inertia which, for those calculations, shall be computed on the assumption that the member is homogeneous, neglecting the reinforcement but including that portion of the concrete section outside of the reinforcement which is ordinarily considered as fireproofing. l and h are the span length and column height respectively, as defined.

(C) Continuous beams or slabs of unequal span or with non-uniform loads.

Continuous beams with substantially unequal spans, or with other than uniformly distributed loading, whether freely supported or restrained, should be designed for the maximum moments resulting from the most severe probable combination of loading and restraint.* Provision should be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

(D) Where it is necessary to introduce steel in compression in girders, beams, or slabs, such steel should be thoroughly anchored by ties or stirrups not less than one-fourth inch (¼″) in size, which should be spaced not more than eight inches (8″) apart over the distance where the compression steel is required.

(E) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement should be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement should provide for the following minimum ratios of reinforcing area to concrete area. In no case should such reinforcing bars be placed farther apart than five (5) times the slab thickness nor more than eighteen inches (18″):**

Ratio of Steel Area to Concrete Area

	Floor Slabs	Roof Slabs
Plain Bars	0.0025	0.003
Deformed Bars	0.002	0.0025

Shear and Web Reinforcement

Symbols

A_v = total area of web reinforcement in tension in a section, or the total area of all bars bent up in any one plane.

α = angle between inclined web bars and axis of beam.

f'_c = ultimate compressive strength of concrete at age of 28 days.

f_v = tensile unit stress in web reinforcement.

s = spacing of stirrups measured perpendicular to the direction of the stirrups.

v = shearing unit stress.

V = total shear.

V' = excess of total shear over that permitted on the concrete.

All other symbols as defined in (1) to (4).

For approximate results in the following formulas it may be assumed that $j=7/8$.

*In those cases, where end restraint is assumed for the purpose of reducing the positive moment, it is advisable to assume that the sum of the positive and negative moments should equal:

$$\frac{6}{5}\times\frac{wl^2}{8}$$

**It will often be possible to calculate the longitudinal reinforcement in beams, when beam and slab construction is used, as being effective as temperature reinforcement in the adjoining slab.

Beams or Slabs

Since shearing stresses are a convenient measure of diagonal tension which must be provided for, the provisions of this section are expressed in terms of shear. The shearing unit stress, v, in reinforced concrete beams is to be computed by formula 29:

$$v = \frac{V}{bjd} \qquad (29)$$

When the value v exceeds the allowable unit shearing stresses as given on page 38, web reinforcement should be provided to carry the excess.

For beams of I or T section, b' shall be substituted for b in formula 29.

In tile and joist construction, b may be taken as a width equal to the thickness of the concrete web plus the thickness of the vertical webs of the concrete or clay tile in contact with the joist.

Web reinforcement may consist of:

(a) Vertical stirrups or web reinforcing bars.

(b) Inclined stirrups or web reinforcing bars forming an angle of thirty degrees (30°) or more with the axis of the beam.

(c) Longitudinal bars bent up at an angle of fifteen degrees (15°) or more with the axis of the beam.

Stirrups or bent up bars to be considered effective as web reinforcement should be specially anchored at both ends.

Area of steel required in stirrups is to be computed by formula 30:

$$A_v = \frac{V's}{f_v jd} \qquad (30)$$

Where the shearing stress is not greater than $0.06f'_c$, the distance, s, between two successive stirrups measured perpendicular to the direction of the stirrup should not exceed $\frac{3}{4}d$, and where unit shearing stress exceeds $0.06f'_c$, it should not be greater than $\frac{3}{8}d$.

Where there is a series of parallel bent up bars at varying distances from the support they should be designed as inclined stirrups according to formula 30.

Where bent up bars in a single plane are used for web reinforcement, the required area of the bar is to be computed by formula 31:

$$A_v = \frac{V'}{f_v \sin\alpha} \qquad (31)$$

In formula 31, V' shall not exceed $0.035f'_c bd$, nor α be less than fifteen degrees (15°). Only the center three-fourths ($\frac{3}{4}$) of the inclined portion of such bars or group of bars is to be considered effective in resisting shear. Between the face of the support and the area reinforced by the bent-up bars, other web reinforcement should be provided for beams carrying concentrated loads and, unless the distance is less than $\frac{1}{2}d$, for beams carrying only uniform load.

Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam may be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete should be included only once.

Flat Slabs

In flat slabs, the shearing unit stress computed by formula 29 (in which d, shall be taken as $t_1 - 1\frac{1}{2}$) on a vertical section which lies at a distance $t_1 - 1\frac{1}{2}$, from the edge of the column capital and parallel with it, should not exceed $0.03f'_c$, when at least fifty per cent (50%) of the total negative reinforcement in the column strip passes directly over the column capital. The unit shearing stress should not exceed $0.025f'_c$, when twenty-five per cent (25%) of the total negative reinforcement passes directly over the column capital (which is the least that should be permitted). For intermediate percentages, intermediate values of the shearing unit stress should be used.

The shearing unit stress computed by formula 29 (in which d, is taken as $t_2 - 1\frac{1}{2}$), on a vertical section which lies at a distance $t_2 - 1\frac{1}{2}$, from the edge of the dropped panel and parallel with it should not exceed $0.03f'_c$. At least fifty per cent (50%) of the cross-sectional area of the negative reinforcement in column strip must be within the width of strip directly above the dropped panel.

Footings

The shearing unit stress computed by formula 29, on a vertical section, which lies at a distance d, from the face of the supported column or pier and parallel with it, should not exceed $0.02f'_c$, for footings with straight bars, nor $0.03f'_c$, for footings in which the bars are specially anchored at both ends by adequate hooks or as otherwise specified.

In footings supported on piles, the critical section for diagonal tension is to be considered the distance $d/2$ from the face of the column or pedestal and any piles whose centers are at or within this section should be excluded in computing the shear.

Bond and Anchorage

Symbols

o = circumference or perimeter of bar.
Σ_o = sum of perimeters of bars in one set.
u = bond stress per unit of area of surface of bar.

All other symbols as previously defined.

For approximate results in the following formula it may be assumed that $j = \frac{7}{8}$.

Where bar reinforcement is used to resist tensile stress developed by beam action, the bond stress is to be taken as not less than that computed by formula 32.

$$u = \frac{V}{jd\Sigma_o} \qquad (32)$$

For continuous or restrained members, the critical section for bond for the positive reinforcement should be assumed to be at the point of inflection, that for the negative reinforcement should be assumed to be at the face of the support, and at the point of inflection. For simple beams or at the outer ends of freely supported end spans of continuous beams, the critical section for bond should be assumed to be at the face of the support.

Bent-up longitudinal bars which, at the critical section, are within a distance $d/3$ from horizontal reinforcement under consideration may be included with the straight bars in computing Σ_o.

Ordinary Anchorage Requirements

For ordinary anchorage requirements, tensile negative reinforcement in continuous restrained or cantilever beams should have a length of anchorage beyond the face of the support sufficient to develop the full maximum tension at an average bond stress not greater than $0.04f'_c$, for plain bars or $0.05f'_c$, for deformed bars. In continuous or restrained beams, negative reinforcement should be carried to or beyond the point of inflection.

Of the positive reinforcement in continuous beams, not less than one-fourth (¼) the area should extend at the same face of the beam into the support to provide an embedment of ten (10) or more bar diameters.

For non-continuous beams not less than one-half (½) of the area of positive reinforcement should extend at the same face of the beam into the support to provide an embedment of ten (10) or more bar diameters.

Special Anchorage Requirements

Where increased shearing or bond stresses on account of special anchorage are permitted, special anchorage of all reinforcement, in addition to the ordinary anchorage previously described, should be provided as follows:

(a) In continuous and restrained beams, anchorage beyond points of inflection of at least one-third (⅓) the area of the negative reinforcement and beyond the face of the support of at least one-third (⅓) the area of the positive reinforcement, shall be provided to develop one-third (⅓) of the maximum working stress in tension, at average bond stresses not to exceed $0.04f'_c$, for plain nor $0.05f'_c$, for deformed bars.

(b) At the edges of footings, all the bars shall extend along the tension face to a point three inches (3″) from the edge of the footing and be anchored by a hook providing an additional length of at least twelve (12) bar diameters.

(c) In simple beams or at the outer ends of freely supported end spans of continuous beams, at least one-half (½) of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third (⅓) of the maximum working stress in tension, at an average bond stress not to exceed $0.04f'_c$, for plain bars nor $0.05f'_c$, for deformed bars.

Web bars should be anchored at both ends by one of the following methods or combination thereof. Only anchorage meeting the requirements of (a), (b) and (c) shall be used for shearing unit stresses in excess of $0.08f'_c$.

(a) providing continuity with the longitudinal reinforcement; or

(b) bending around the longitudinal bar or steel shape; or

(c) a hook which has a radius of bend not less than four (4) times the diameter of the web bar.

(d) a length of embedment sufficient to develop the stress in the stirrup by bond as provided below, provided the other end of the stirrup is anchored as in (a).

The end anchorage of a web member not in bearing on the longitudinal reinforcement should be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups should be carried as close to the upper and lower surfaces as fireproofing requirements permit.

The stress in a stirrup or web reinforcement bar should not exceed a value equal to the surface area of the bar embedded within the upper or lower one-half (½) of the beam multiplied by $0.04f'_c$, for plain bars or $0.05f'_c$, for deformed bars, and in no case more than 16,000 lbs. per square inch.

Flat Slabs

Symbols

c = diameter, in feet, of column capital at the under side of the slab or dropped panel. No portion of the column capital may be considered for structural purposes, which lies outside of the largest ninety degree (90°) cone that can be included within the outlines of the column capital.

l = span length in feet of the flat slab panel, center to center of columns in the direction in which moments are considered.

M_o = sum of positive and negative bending moments, at the principal design sections, in the direction in which the length is given by l. This moment is in foot pounds where the other items are in the units indicated below.

t_1 = thickness of flat slab without dropped panels.

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel.

w' = uniformly distributed dead and live load per unit of area of a floor or roof.

W = total dead and live load uniformly distributed over a single panel area.

All other symbols as previously defined.

The term flat slabs as used in this section, refers to concrete slabs, having reinforcement bars extending in two or four directions, without beams or girders to carry the load to supporting members.

The moment coefficients, moment distribution, and slab thicknesses specified herein are for slabs, which have three (3) or more rows of panels in each direction, and in which the panels are approximately uniform in size.

Slabs with paneled ceiling or with dropped panels may be considered as coming under the requirements herein given, provided the dropped panel has a length or diameter in each direction parallel to a side of the panel of not less than 0.35 of the panel length in that direction, and provided further that the depth of the thicker portions of the slab does not exceed one and one-half ($1\frac{1}{2}$) times the depth of the remainder of the slab. This section does not apply to flat slabs in which the ratio of length to width of panel exceeds one and one-third ($1\frac{1}{3}$).

For convenience of reference, a flat slab panel shall be considered as consisting of strips as follows:

(a) A *middle strip* one-half panel in width symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered.

(b) A *column strip* one-half panel in width occupying the two quarter panel areas outside of the middle strip.

When considering moments in the direction of the width of the panel, the panel is similarly divided by strips, the widths of which are each one-half the length of the panel.

The bands of steel reinforcing bars in a four-way flat slab are designated as follows:

(a) A *diagonal band* approximately $0.4l$ in width, symmetrical with respect to the diagonal running from corner to corner of the panel, and extending through the panel in the direction in which moments are being considered.

(b) A *direct band* approximately $0.4l$ in width, symmetrical with respect to the line of centers of supporting columns and extending through the panel in the direction in which moments are being considered.

In the succeeding paragraphs, the provisions for limiting moments, etc. are related to certain critical sections. These sections are referred to as principal design sections and are located as follows:

(a) *Sections for Negative Moment.* These shall be taken along the edges of the panel, on lines joining the column centers, and following the circumference of the column capital.

(b) *Sections for Positive Moment.* These shall be taken on the centerline of the panel.

The numerical sum of the positive and negative moments at the principal design sections in an interior panel in the direction of either side of a rectangular panel should not be less than that given by formula 33.

$$M_o = 0.09Wl\left(1-\frac{2c}{3l}\right)^2 \quad \ldots\ldots\ldots\ldots (33)$$

The moments in the principal design sections shall be those given in the accompanying table of moments except that the maximum negative moment in the column strip may be greater or less than the values given in table of moments by not more than $0.03\ M_o$, provided that the sum of the moments on the principal sections remains equal to M_o, and provided further that the moment in each of the three other critical design sections be modified by not more than $0.01M_o$.

In computing the ratio of reinforcement for negative moment in the column strip, the width of section should be taken as equal to the width of the dropped panel, where used, or a half width of panel where no dropped panel is used.

The width of a band of steel in a two-way system should be such as to provide reinforcement over an entire one-half panel width.

The band width for the direct bands in the four-way system should be approximately four tenths (0.4) of the panel width at right angles to the direction of the band and for diagonal bands approximately four tenths (0.4) of the average span length. In proportioning the reinforcement in this system it should be assumed that reinforcement in the direct band resists the entire positive moment for the column strip and that in the two diagonal bands resists the entire positive moment for the middle strip.

Reinforcement for negative moment for the column strip should include the area of reinforcement for negative moment in the diagonal bands multiplied by the cosine of the angle between the diagonal band and the axis of the direct band considered, plus the full area of the reinforcement for negative moment in the direct band. The negative reinforcement for the middle strip should be provided independently of the diagonal bands.

Special Case.

$c = 0.225l$

For the particular case where c equals 0.225 times the average span length (the average of the distances center to center of columns on the two sides of the panel), the value of M_o may be taken as given by formula 34:

$$M_o = 0.065Wl \quad \ldots\ldots\ldots\ldots (34)$$

Use l as defined on page 33.

For two-way slabs, the value of M_o may be obtained from formula 34 and the distribution taken from the table at the foot of this page.

For the four-way slab with dropped panels, the following table of coefficients may be used in computing the reinforcement required in each of the bands,—

Moments to Be Used in Design of Flat Slabs for Interior Panels Fully Continuous

General Case: all values of c; M_o given by formula 33

STRIP	Flat Slabs Without Dropped Panels		Flat Slabs With Dropped Panels	
	Negative	Positive	Negative	Positive
Slabs with 2-Way Reinforcement				
Column strip	$-M_c = 0.46M_o$	$+M_c = 0.22M_o$	$-M_c = 0.50M_o$	$+M_c = 0.20M_o$
Middle strip	$-M_m = 0.16M_o$	$+M_m = 0.16M_o$	$-M_m = 0.15M_o$	$+M_m = 0.15M_o$
Slabs with 4-Way Reinforcement				
Column strip	$-M_c = 0.50M_o$	$+M_c = 0.20M_o$	$-M_c = 0.54M_o$	$+M_c = 0.19M_o$
Middle strip	$-M_m = 0.10M_o$	$+M_m = 0.20M_o$	$-M_m = 0.08M_o$	$+M_m = 0.19M_o$

provided that l for the direct bands is the center-to-center distance between columns in the direction in which the band extends, and for the diagonal bands the average value of l for the two direct bands of the panel. The moments in the table are those on sections at right angles to the direction of the respective bands:

Band	Location	Moment
Direct	Center	$+0.012Wl$
Diagonal	Center	$+0.009Wl$
Direct	At column head	$-0.020Wl$
Diagonal	At column head	$-0.011Wl$
Top band across direct band	Between columns	$-0.005Wl$

Dimensions

For slabs without dropped panels, using concrete of 2,000 lb. per sq. in. ultimate strength, the total thickness of the slab t_1, in inches, is to be not less than the value given by formula (35).

$$t_1=0.038\left(1-1.44\frac{c}{l}\right)l\sqrt{w'}+1\tfrac{1}{2}\ldots\ldots(35)$$

For slabs with dropped panels, using concrete of 2,000 lb. per sq. in. ultimate strength, the total thickness in inches at points beyond the dropped panel is to be not less than

$$t_2=0.02\,l\,\sqrt{w'}+1\ldots\ldots\ldots\ldots(36)$$

The total slab thickness through the dropped panel should not be greater than $1.5t_2$, nor less than $1.25t_2$. The side or diameter of the dropped panel should not be less than 0.35 times the side of the panel in the parallel direction.

In determining minimum thickness by formulas (35) and (36), the value of l is to be the panel length center to center of the columns, on the long side of the panel. For concrete of 2,000 lb. per sq. in. ultimate strength, the slab thickness t_1 or t_2 should in no case be less than $l/32$ for floor slabs, and not less than $l/40$ for roof slabs.

Where concretes of higher ultimate strengths than 2,000 lb. per sq. in. are used, the thickness given by the formulas (35) and (36) and the limiting thicknesses may be reduced by multiplying by the factor $\sqrt[3]{\frac{2000}{f'_c}}$ in which f'_c is the ultimate strength of the concrete to be used.

The ratio of reinforcement for negative moment in the column strip should not exceed the values of p calculated for balanced reinforcement, that is, the amount of reinforcement for which both the steel and the concrete are stressed to the full amount permitted. Any reinforcement in excess of this amount should not be included in the calculation.

The ratio of flat slab reinforcement in any strip should not be less than 0.0025. Bars should not be spaced farther apart than one and one-half ($1\frac{1}{2}$) times the slab thickness.

Points of Inflection

In the middle strip the point of inflection for slabs without dropped panels should be assumed at a line $0.33l$ distant from the center of the span, and for slabs with dropped panels $0.3l$ distant from the center of the span.

In the column strip, the point of inflection for slabs without dropped panels should be at a line 0.33 $(l-c)$ distant from the center of the panel and 0.3 $(l-c)$ for slabs with dropped panels.

Arrangement of Reinforcement

Provision should be made for securing the reinforcement in place, so as to resist properly not only the critical moments, but also the moments at intermediate sections. The full area of steel required for negative moment at the column capital should be continued in the same plane close to the upper surface of the slab to the edge of the dropped panel, but in no case less than a distance $0.2l$ from the center line of columns. Lapped splices should not be permitted at or near regions of maximum stress.

Two-Way System

Column Strips. At least four-tenths (0.4) of the area of steel required at the section for positive moment in the column strip should be of such length and so placed as to reinforce the negative moment section at the two adjacent column capitals. These bars, and any other bars for negative reinforcement should extend into the adjacent panel to a point at least $0.05l$ beyond the point of inflection. Not less than one-third ($\frac{1}{3}$) of the bars used for positive reinforcement in the column strip should extend into the dropped panel at least twenty (20) diameters of the bar, but not less than twelve inches (12″), or in case no dropped panel is used, should extend to within $0.125l$ of the center line of the columns or the supports. The balance of the bars for positive reinforcement should extend at least $0.33l$ on either side of the center line of panel.

Middle Strips. For the middle strip at least one-half ($\frac{1}{2}$) of the bars for positive moment should be bent up and extend over the main bands at both sides of the panel to a point at least $0.25l$ beyond the center line of columns. The location of the bends should be such that for a distance $0.15l$ for slabs with dropped panels, or $0.125l$ for slabs without dropped panels, on each side of the center line of columns, the full reinforcement required for negative moment will be provided in the top face of the slab. The full reinforcement for positive moment in the middle strip should extend in the bottom face of the slab to a point at least $0.3l$ on either side of the panel center line, and at least fifty per cent (50%) of it should extend to points $0.325l$ on either side of the panel center line for slabs with dropped panels, or $0.35l$ for slabs without dropped panels.

Four-Way System

Direct Bands. All provisions governing the placing of steel in column strips in two-way systems apply as well to the direct bands in four-way systems.

Diagonal Bands.—At least four-tenths (0.4) of the area of steel required at the section for positive moment should be of such length and so placed as to reinforce the negative moment section at the two diagonally opposite column capitals. These bars and any other bars for negative reinforcement should extend into the adjoining panel to points at least $0.4l$ beyond a line drawn through the column center perpendicular to the direction of the band. The straight bars for positive moment in the diagonal bands should not be shorter than the longer straight bars in the direct bands.

Negative Reinforcement in Middle Strips. The steel required for negative moment in the middle strip should extend not less than $0.25l$ on either side of the column center lines.

Wall Panels

In wall panels and other panels in which the slab is non-continuous on one edge, the maximum positive moments on the principal design section parallel to the edge should be increased by twenty-five per cent (25%).

The bars used for positive moments perpendicular to the discontinuous edge should extend to this edge of the panel. All top bars should be bent or hooked to provide adequate bond resistance and the bottom bars should have an embedment of at least six inches (6″) in spandrel beams or columns.

At the wall or discontinuous edge the negative moment in the column strip should be taken as not less than ninety per cent (90%) and in the middle strip not less than sixty-two and five tenths per cent (62.5%) of the corresponding moments for a normal interior panel as given in the table.

Where there is a beam or a bearing wall at the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall should be taken as thirty per cent (30%) greater than the negative moment specified for a middle strip. The half column strip adjacent and parallel to and lying on either side of the beam or wall should be designed to resist moments at least one-fourth of those specified for a column strip. The beam or wall in such cases should be designed to carry a uniformly distributed load equal to one-fourth (¼) of the panel load on either side in addition to the loads directly imposed upon it.

In panels having a marginal beam on one edge or on each of two adjacent edges, the beam should be designed to carry at least the load superimposed directly upon it, exclusive of the panel load. A marginal beam, which has a depth greater than the thickness of the dropped panel into which it frames, should be designed to carry, in addition to the load superimposed directly upon it, a uniformly distributed load equal to at least one-fourth (¼) of the total live and dead load for which the adjacent panel or panels are designed. Slabs supported by marginal beams on opposite edges should be designed as freely supported slabs for the entire load.

The half column strip adjacent to and parallel with marginal beams having a depth equal to or less than the thickness of the dropped panel should be designed to resist half the moment specified for the column strip.

Where brackets, the faces of which make an angle with the face of the column projected upward of not more than forty-five degrees (45°), are used in place of capitals in wall panels having exterior columns, the value of c, in the direction in which the bracket extends may be taken as twice the distance from the center of the column to a point where the structural portion of the bracket is one and one-half inches (1½″) thick, and averaged with the value of c, for an interior column capital in the computations for moment in formula 33. The value of c, for column strips parallel and adjacent to a non-continuous edge of a slab where either no marginal beam is used, or where the beam used is no deeper than the dropped panel, should be taken as equal to the width of the wall column if no bracket is used in this direction.

The value of c, for column strips parallel and adjacent to marginal beams having a depth greater than the thickness of the dropped panel, should, if no bracket is used in this direction, be taken as equal to the width of the wall column plus twice the difference between the depth of the beam and the depth of the slab through the dropped panel. This value of c is to be used in calculating the $-M_c$ and $+M_c$ for the half column strip parallel and adjacent to the marginal beams only. This half column strip should be designed to resist a moment at least one-fourth (¼) as great as that specified for a column strip in the Table of Moments.

In slabs where dropped panels are used at all interior columns, the dropped panels may be omitted at wall columns provided the panel design complies with the moment and reinforcement percentage requirements for designing slabs without dropped panels.

Columns

Symbols

A = total net area of column or pedestal, exclusive of fireproofing.

A_c = area of core of spirally-hooped column, measured to the outside diameter of the spiral.

A_g = gross area of column with lateral ties.

f_c = safe unit axial compressive stress in concrete in columns.

P = total safe axial load on a column when h/R is less than forty (40).

R = least radius of gyration of a section.

All other symbols as previously defined.

Unless designed as long columns as later provided, reinforced concrete columns should not be longer than eleven (11) times the least lateral dimensions. Principal columns in buildings should have a minimum diameter or thickness of twelve inches (12″). Posts that are not continuous from story to story should have a minimum diameter or thickness of six inches (6″).

The unsupported length of reinforced concrete columns shall be taken as:

(a) In flat slab construction the clear distance between the floor and the under side of the capital;

(b) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level;

(c) In floor construction with beams in one direction only, the clear distance between floor slabs;

(d) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut, respectively, is not less than seventy-five degrees (75°) nor more than one hundred and five degrees (105°).

When reinforced concrete brackets are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by the depth of the bracket, provided the bracket width is at least equal to that of the beam and not less than one-half (½) that of the column.

Spiral Columns

The safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core, should not be greater than that determined by formula 39.

$$P = A_c[1+(n-1)p]f_c \ldots\ldots\ldots (39)$$

in which the values of f_c are as found on page 38 or by the formula:

$$f_c = [300+(0.10+4p)f'_c]$$

The longitudinal reinforcement should consist of at least six (6) bars of minimum diameter of one-half inch (½″) and its effective cross-sectional area should not be less than 0.01, nor more than 0.06 of that of the core. The amount of vertical steel concentrated in any one ring should not exceed 0.04 of the core area. The inner ring if used should be stayed at intervals of twenty-four inches (24″) and should not be nearer than two-tenths (0.2) times the core diameter to the outer ring. Splices in longitudinal reinforcement should provide a lap of at least twenty-four (24) bar diameters for deformed bars and thirty (30) diameters for plain bars.

The ratio of the spiral reinforcement should be not less than one-fourth (¼) the ratio of the longitudinal reinforcement. It should consist of evenly spaced continuous spirals held firmly in place and true to line. At the ends of all spirals and at points of splice of spiral wire, the outside diameter must be maintained. The spacing of the spirals should not be greater than one-sixth (1/6) of the diameter of the core and in no case more than three inches (3″).

Reinforcement should be protected everywhere by a covering of concrete, cast monolithic with the core, which should have a minimum thickness of one and one-half inches (1½″).

Tied Columns

The safe axial load on columns reinforced with longitudinal bars and separate lateral ties should not be greater than that determined by formula 40.

$$P = f_c[A_g+(n-1)A_s] \ldots\ldots\ldots\ldots (40)$$

The amount of longitudinal reinforcement should not be less than 0.005 nor should the amount considered in the calculations be more than 0.02 of the total area of the column. The longitudinal reinforcement should consist of not less than four (4) bars of a minimum diameter of five-eights inch (⅝″) placed at a clear distance from the face of the column not less than two inches (2″). Splices in longitudinal reinforcement should provide a lap of at least twenty-four (24) bar diameters for deformed bars and thirty (30) diameters for plain bars.

Lateral ties should be at least one-fourth inch (¼″) in diameter spaced not more than twelve inches (12″) apart. In columns of rectangular section cross ties should be arranged to afford support to the vertical bars at intervals not greater than the shorter side of the section, but such interval need not be less than twelve inches (12″) in any case.

The bending moments in interior and exterior columns should be determined on the basis of loading conditions and end restraint, and should be provided for in the design.

In flat slab construction, the least dimension of any column should be not less than one-fifteenth (1/15) of the average center to center span, nor less than sixteen inches (16″). For known eccentric loads or unequal spacing of columns, computations of moments in columns should be made accordingly. Wall columns in flat slab construction should be designed to resist a bending moment of $\frac{Wl}{35}$. Any counter moment due to the weight of the structure that projects beyond the column center line may be deducted from the moment computed as just described. Resistance to the bending moments may be divided between the columns immediately above and below in direct proportion to the values of their ratios of I/h.

The recognized methods should be followed in calculating the stresses due to combined axial load and bending. The column section should not be less than that required where axial load alone is considered. The limiting combined unit stresses should be as follows:

(a) Columns with spiral reinforcement.
$[300+(0.10+4p)\,f_c']+0.15f'_c$.

(b) Columns with lateral ties $0.3f'_c$. The total amount of reinforcement considered in the computations shall be not more than four per cent (4%) of the total area of the column.

(c) Tension in longitudinal reinforcement due to bending on the column should not exceed 16,000 pounds per square inch.

Long Columns

The permissible working load on the core in axially loaded spiral columns which have a length greater than fifty times the least radius of gyration of the column core (50R) should not be greater than that determined by formula 41:

$$\frac{P'}{P} = 1.50 - \frac{h}{100R} \ldots\ldots\ldots\ldots (41)$$

where P' = total safe axial load on long columns;
P = total safe axial load on column of the same section whose h/R is less than forty (40), as determined.
R = least radius of gyration of column core.

The permissible working load on axially loaded tied columns which have a length greater than forty times the least radius of gyration of the column core ($40R$) should be not greater than that determined by formula 42.

$$\frac{P'}{P} = 1.33 - \frac{h}{120R} \ldots\ldots\ldots\ldots (42)$$

The radius of gyration of a column shall be computed from the concrete area used in design and the transformed section of the longitudinal steel area, that is, the actual area of steel multiplied by n.

Unit Stresses

The unit stresses in pounds per square inch on the concrete to be used in design shall not exceed the following values, where f'_c equals the minimum ultimate compressive strength at 28 days.

DESCRIPTION	ALLOWABLE UNIT STRESSES			
	For Any Strength of Concrete as Fixed by Test	When Strength of Concrete is Fixed by the Water-Cement Ratio		
		$f'_c=$ 2000 lb.	$f'_c=$ 2500 lb.	$f'_c=$ 3000 lb.
Flexure:				
Extreme fiber stress in compression	$0.40f'_c$	800	1000	1200
Extreme fiber stress in compression adjacent to supports of continuous or fixed beams	$0.45f'_c$	900	1125	1350
Shear:				
Beams with no web reinforcement and without special anchorage of longitudinal steel	$0.02f'_c$	40	50	60
Beams with no web reinforcement, but with special anchorage of longitudinal steel	$0.03f'_c$	60	75	90
Beams with properly designed web reinforcement, but without special anchorage of longitudinal steel	$0.06f'_c$	120	150	180
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	$0.09f'_c$	180	225	270
Flat slabs at distance d from edge of column capital or dropped panel	$0.03f'_c$	60	75	90
Footings where longitudinal bars have no special anchorage	$0.02f'_c$	40	50	60
Footings where longitudinal bars have special anchorage	$0.03f'_c$	60	75	90
Bond:				
In beams and slabs and one-way footings:				
Plain bars	$0.04f'_c$	80	100	120
Deformed bars	$0.05f'_c$	100	125	150
In two-way footings:				
Plain bars	$0.03f'_c$	60	75	90
Deformed bars	$0.0375f'_c$	75	94	112
Where special anchorage is provided, double these values in bond may be used.				
Bearing:				
Where a concrete member has an area at least twice the area in bearing	$0.25f'_c$	500	625	750
Axial Compression:				
In columns with lateral ties	$0.225f'_c$	450	563	675
In columns with continuous spirals enclosing a circular core:				
Ratio of longitudinal reinforcement $p=0.01$	$[300+0.14f'_c]$	580	650	720
0.02	$[300+0.18f'_c]$	660	750	840
0.03	$[300+0.22f'_c]$	740	850	960
0.04	$[300+0.26f'_c]$	820	950	1080
0.05	$[300+0.30f'_c]$	900	1050	1200
0.06	$[300+0.34f'_c]$	980	1150	1320
(Spiral reinforcement not to be less than ¼ the longitudinal).				

The following unit stresses in steel reinforcing shall not be exceeded:

Tension:

Intermediate grade billet steel .... 20,000 lb. per sq. in.
Web reinforcement, hangers or other direct tension members .... 16,000 lb. per sq. in.

Compression:

Intermediate grade billet steel .... nf_c

BOOK III

A Load Survey of a Reinforced Concrete Warehouse

List of Illustrations

Note: The first floor is the ground floor and it did not need to be investigated.

PLATE I

PLATE II

PLATE III

PLATE IV

PLATE V

PLATE VI

Copy of the Report sent to the Georgia Power Company's Purchasing Agent by Mr. H. H. Terhune of the Allied Engineers, Inc., head of the Building Department.

"SUBJECT: 211 DECATUR STREET - LOADING SURVEY.

Mr. F. A. Jordan, Purchasing Agent,

Georgia Power Company,

Building.

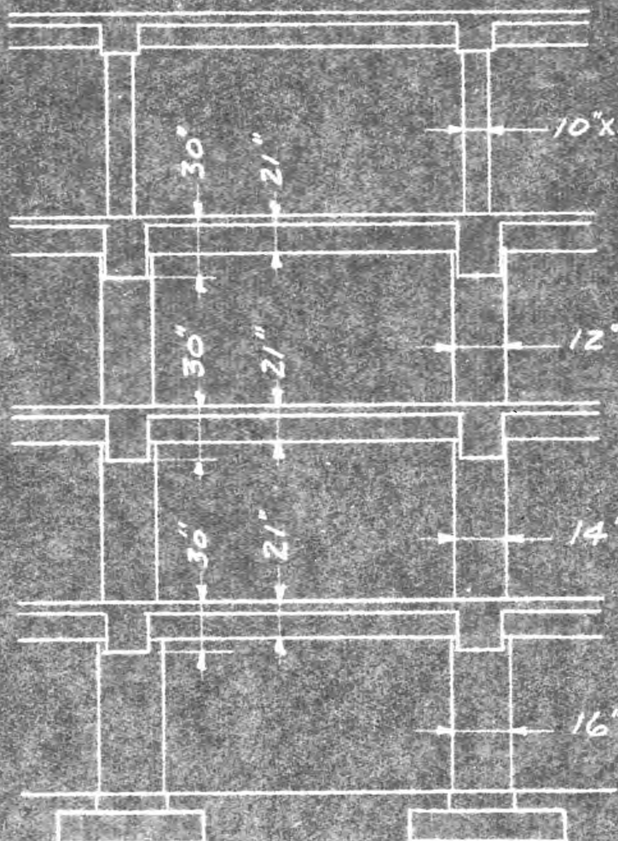
Dear Sir:

In accordance with request made by your Mr. R. L. Leach, Mr. H. H. Terhune of this department has made a survey of the concrete portion of the Warehouse at 211 Decatur Street, Atlanta, Georgia.

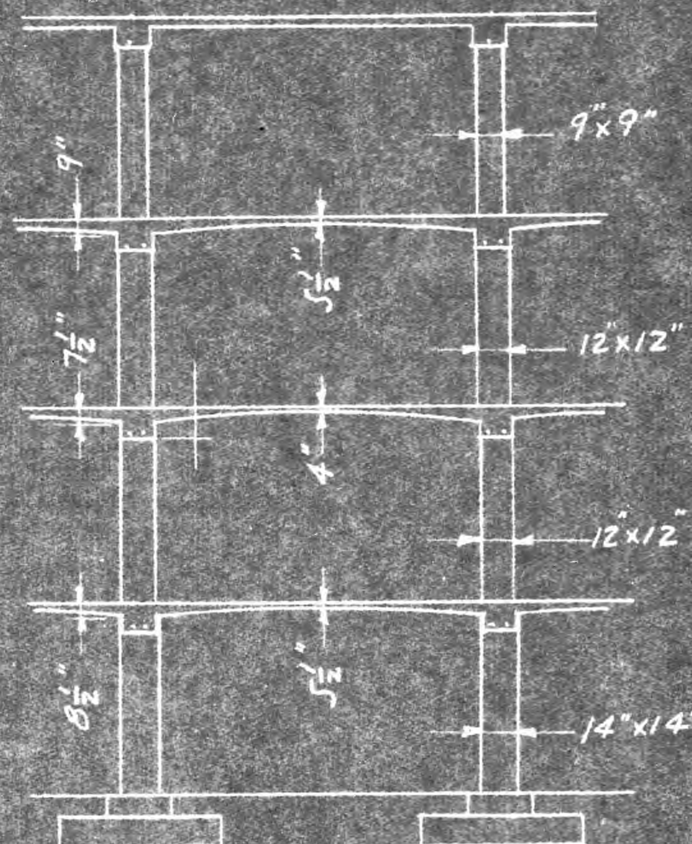
From records available, it has been found that this building was erected about April 1907, being designed by the Southern Ferro Concrete Company, their job #19. The original design as now on file at the office of Building Inspector, City Hall, Atlanta, shows this building was designed as a girder, beam and slab type of structure, whereas the present construction is of beam and slab type. This change in design materially decreases the allowable live load which may be superimposed upon floor. The attached sketch shows in detail a typical bay; first as the building was originally designed and then as it is now constructed.

As there was no information obtainable either from the constructors or files at City Hall relative to specifications, requirements, design engineering, etc., and as the meager information which is on file at the City Hall does not show the materials used in the reinforced concrete structure, it was necessary to break into the concrete to ascertain just what reinforcing material was used. The concrete was cut into at random places at zero bending moment points and sizes of steel obtained by micrometer. At points obtained on the second and fourth floors and roof, only

CROSS SECTION OF GA. POWER CO. WAREHOUSE
#211 DECATUR ST.



AS SHOWN ON BLUEPRINT
OF SOUTHERN FERRO CONCRETE CO.



AS TAKEN FROM BUILDING
AS IT STANDS TODAY.

one steel bar was found in the beam, this being one $1\frac{1}{4}$" diameter for the second floor and fourth floor and one 5/8" diameter for roof. The reinforcing in floor slabs varied from 3/8" diameter bars 3" on centers to 10" on centers. It was found, however, where the beam was opened up on the third floor that the reinforcing in this beam consisted of two $1\frac{1}{4}$" diameter bars.

An accurate set of levels were taken from the fourth floor to the first to obtain the thickness of the floor slab at spring line and at center line of slab, this being an arched type of floor. As the thickness at the various points in the several floors varied, the average thickness of readings taken was used in checking over the design. A sample of the concrete in the beam was obtained and tested in the Materials Laboratory of Georgia Tech. This concrete tested at 2800# per sq. in., which would indicate a weak mix concrete, as same is now about twenty-three years old and over such a long period of time the strength of the concrete should have increased.

The following factors controlling design of concrete structures in the City of Atlanta is taken from the Building Code as follows, while the Joint Committee of American Society of Testing Materials and American Society of Civil Engineers, etc. recommendations are set opposite same as a comparison:

City of Atlanta Building Code	Joint Committee of American Society of Testing Materials, Etc.
Allowable fibre stress in concrete	
750#/Sq. In.	1000#/Sq. In.
Allowable shearing stress no stirrups	
30#/Sq. In.	50#/Sq. In.

Allowable shearing stress with stirrups

50#/Sq. In.	75#/Sq. In.

Allowable bond stress

80#/Sq. In.	100#/Sq. In.

Allowable stress in steel

16,000#/Sq. In.	18,000#/Sq. In.

In order to take advantage of the test made on concrete and to utilize as high a working stress as possible, and still be within the recommended limits of present day design, the unit stresses of the Joint Committee were used.

In checking design certain assumptions were made, these assumptions being based on engineering data available and used at time this structure was built. For a working basis, as it was impossible to open up every concrete beam, it was necessary to assume that the reinforcing bars within the beam were bent up at quarter points and carried over point of support, thus giving the same cross-sectional area of steel at positive, as well as negative bending points. On this basis the following available live loads were obtained:

Fourth Floor

Slab - 213#/Sq. Ft.

Beam - 210#/Sq. Ft.

Second Floor

Slab - 304#/Sq. Ft.

Beam - 96#/Sq. Ft.

Third Floor

Slab - 125#/Sq. Ft.

Beam - 103#/Sq. Ft.

Investigation of the loadings on the various floors was made by the Engineering Department in company with one of the men in the Stock Room who gave information as to the actual weight of the various materials. Several panels appeared to be overloaded and these panels were checked over with the following results:

1 - <u>Fourth Floor - Meter Dept. Warehouse</u>

Panel #50 shows 132#/Sq. Ft.

2 - <u>Third Floor</u>

Panel #24 shows a 140# to the sq. ft; panel #25, 193# to the sq. ft.; panel #28 shows 230# to the Sq. Ft.

3 - <u>Second Floor</u>

Where reels of cable and wire are stored it has been found that the floor is now loaded to the maximum and we recommend that no additional reels be placed within the column areas. We also recommend that the reels be not stacked one upon the other.

4 - <u>Roof</u>

Figures show that the roof panels are stressed up to the design unit stresses. Therefore, we recommend that the crane hoist which is hanging from the roof beam and which is used by the Meter Repair Department be removed and other means of supporting same be provided if this crane hoist is necessary.

It will be noted from the preceding figures that all our computations have been based on the maximum allowable stresses used in good engineering practice of today. Had we used the stresses as recommended by the Building Code of the City of Atlanta the loadings on the floors would have been materially decreased. Also we have used area of two steel reinforcing bars in all beam computations, whereas it shows only one bar in several panels. The reason for using two bars in computations is that it is very apparent the building was designed for the allowable loads which two bars give. On the assumption of only one bar in beams, as was found in the second and fourth floor beams, the allowable live load would amount to only 60# per sq. ft.

It was noted that where the various floors showed signs of cracking that this cracking occurred over the beams or perpendicular to same between columns. This would indicate that at the time this building was constructed the theory of continuous bending moments were little known of in the design of concrete structures and that these stresses were not taken care of by the proper bending up of, and the placing of the reinforcing steel. Concrete slab also shows some disintegration and cracking where considerable trucking had been taking place.

After making this investigation and checking over the loadings on the various floors, we recommend that this building be posted for a live load not to exceed 100# per sq. ft., or an equivalent of one-half that amount concentrated at the center of any one beam. The reason we hesitate in recommending increasing the live load over 100# per sq. ft. is that we have had to make a lot of assumptions as stated in this investigation.

We are also advising the Construction Department to patch up the beams and slabs where it is was necessary for us to cut into same to obtain the sizes of reinforcing steel.

Attached herewith find prints showing outline of construction of the above concrete structure and also print referring to the bays overloaded.

On April 7 we sent Mr. Leach report of the loadings on the mill constructed portion of the warehouse at 211 Decatur Street, and advised in that letter that several of the girders supporting the various floors were badly checked and that same be reinforced. As we do not know what disposition has been made of this matter we are again calling this to your attention and if you so desire we will be glad to make investigation of these defective beams, prepare estimate and request authority for putting same in proper condition.

District Engineer "

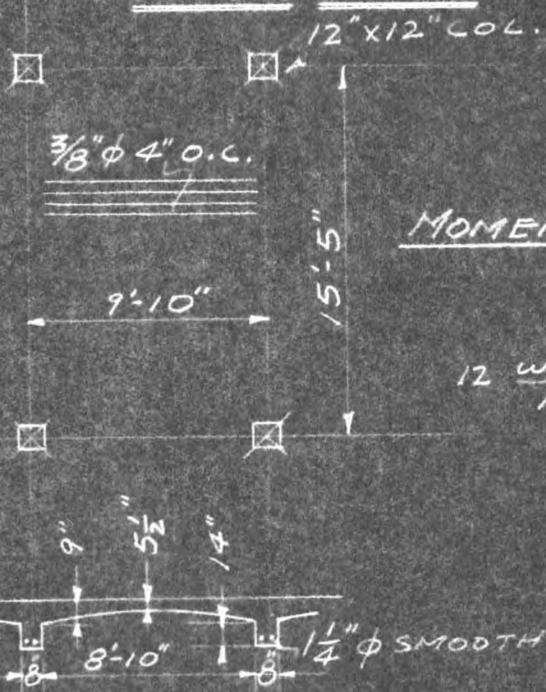
LOAD SURVEY OF GA. POWER CO. WAREHOUSE
LOCATED AT 211 DECATUR ST. ATLANTA, GA.

JOINT COMMITTEE RECOMMENDATIONS.

$f_c' = 2820$ #/□" ACTUAL - USED 2500 #/□"
$f_c = 1000$ #/□"
$k = 0.4$
$j = .867$ (7/8 WAS USED)
$p = .0111$
$K = 172.3$
$v = 50$ #/□" NO SPEC. ANCH. NO WEB STEEL
$v = 75$ #/□" WITH " " " " "
$v = 150$ #/□" NO " " WITH " "
$u = 100$ #/□" PLAIN SMOOTH BARS.
$n = 12$

FOURTH FLOOR

FLOOR CHECK



TYPICAL PANEL

CONSTANTS

$l = 8.83'$; $A_s = (.399)^2 \frac{\pi}{4} = .125$□"
$\Sigma_0 = 1.255"$; $d = 5"$ AT ℄
$d = 8"$ AT BEAM; $M = \frac{wl^2}{10}$

MOMENT ℄

$M = K b d^2$ IF BALLANCED

$$12 \frac{wl^2}{10} = 172 \times 12 \times 5^2$$

$$w = \frac{172 \times 12 \times 25 \times 10}{8.83^2 \times 12}$$

$w = 552$ #/□'

$$M = A_s f_s j d$$

$$w = \frac{3 \times .125 \times 18000 \times .867 \times 5 \times 10}{12 \times 8.83^2}$$

$w = 313$ #/□' = MINIMUM

SHEAR

$$V = bdvj = \frac{wl}{2} = 4.42w$$

$$w = \frac{12 \times 8 \times 50 \times .867}{4.42} = 943 \,\#/\square'$$

BOND

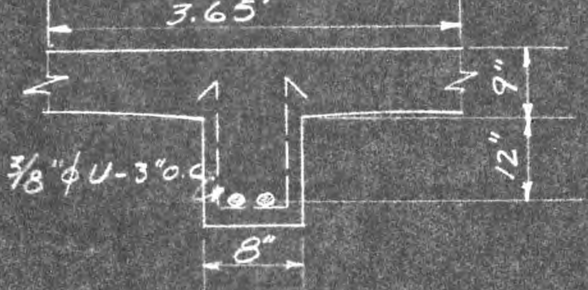
$$V = \Sigma o\, u\, j\, d = 4.42w$$

$$w = \frac{3 \times 1.255 \times 100 \times .867 \times 8}{4.42} = 590 \,\#/\square'$$

$w = D.L. + L.L.$ Av. $D.L. = 100 \,\#/\square'$

$\therefore$ ALLOWED L.L. ON FLOOR = $313 - 100 = 213 \,\#/\square'$

BEAM CHECK



CONSTANTS

$l = 14.58'$; $A_s = 2 \times 1.23 = 2.46 \square''$

$$b = \frac{14.58}{4} = 3.65'$$

CHECK AS "T" BEAM

$$\frac{f_c}{f_s} = \frac{1000}{18000} = .0555$$ NO INTERSECTION ON "T" BEAM CURVE

$\therefore$ N.A. IN FLANGE & IT IS NOT A "T" BEAM

CHECK AS RECTANGULAR BEAM

MOMENT

$$12\,\frac{14.58^2\,w}{10} = 2.46 \times 18000 \times .867 \times 21$$

$$w = 3160 \,\#/\text{LIN. FOOT}$$

$$12\,\frac{14.58^2\,w}{10} = 172 \times 3.65 \times 12 \times \overline{21}^2$$

$$w = 13000 \,\#/\text{LIN. FOOT}$$

SHEAR

$V = V_c + V_s = \frac{wl}{2}$ (AT COLUMN)

$V_c = 50 \times 21 \times 8 \times .867 = 7300^{\#}$

$V_s = \frac{A_s f_s j d}{S} = \frac{2 \times .11 \times 18000 \times .867 \times 21}{3}$

$= 24000^{\#}$

$V = 31300^{\#}$

$w = \frac{31300 \times 2}{14.58} = 4300^{\#}$/LIN. FOOT

CANNOT CHECK BOND-IMPOSSIBLE TO GET TO STEEL AT COLUMNS WITHOUT DANGERING BUILDING.

DEAD LOAD

SLAB = $9.83 \times 1 \times \frac{8}{12} \times 150 =$ $983^{\#}$/L.F.

BEAM = $\frac{14}{12} \times \frac{8}{12} \times 1 \times 150 =$ $117^{\#}$/L.F.

TOTAL D.L. = $1100^{\#}$/LIN. FT.

$\therefore$ ALLOWED L.L. $= 3160 - 1100 = 2060^{\#}$/LIN. FT.

$= \frac{2060}{9.83} = 210^{\#}/\square'$

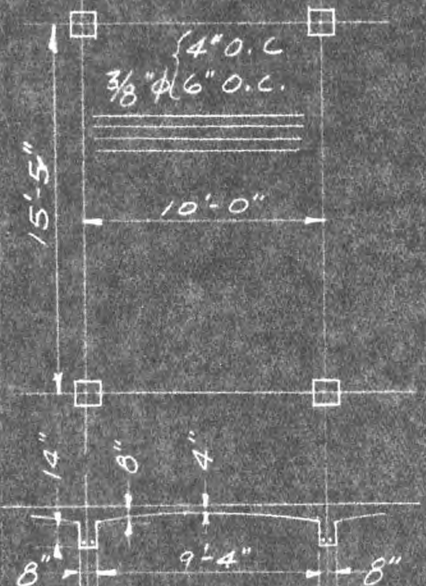
THE STEEL IN THE BEAM WAS ASSUMED TO BE TWO $1\frac{1}{4}''\phi$ EVEN THO. BUT ONE WAS FOUND WHEN WE OPENED IT. REASON TELLS US THAT THIS IS THE EXCEPTION SO WE USE TWO.

WHEN BUT ONE BAR IS USED IN OUR COMPUTATIONS THE ALLOWED LIVE LOAD = $50^{\#}/\square'$

THIRD FLOOR

FLOOR CHECK

COLUMNS ON AV. = 12"×12"



TYPICAL PANEL

CONSTANTS

$l = 9.33'$; $A_s = (.398)^2 \frac{\pi}{4} = .125 \square''$

$\Sigma_o = 1.25''$; $d = 3\frac{1}{2}''$ AT ℄

$d = 7''$ AT BEAM; $M = \frac{wl^2}{10}$

MOMENT ℄ 6" O.C.

$$\frac{12\, w\, 9.25^2}{10} = 2 \times .125 \times .867 \times 3.5 \times 18000$$

$$w = 133 \,\#/\square'$$

∴ 4" O.C. SPACING

$$w = \tfrac{3}{2}(133) = 200 \,\#/\square'$$

SHEAR $\quad \frac{w \times 9.25}{2} = 12 \times 7 \times .867 \times 50$

$$w = 788 \,\#/\square'$$

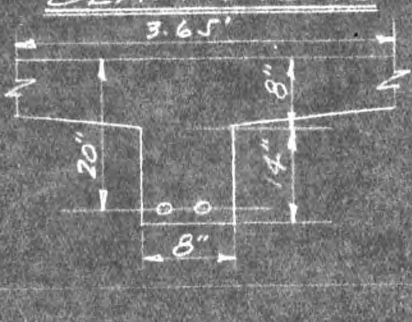
BOND $\quad \frac{w \times 9.25}{2} = 3 \times 1.25 \times 100 \times .867 \times 7$

$$w = 496 \,\#/\square'$$

AV. D.L. = 75 #/□' ∴ ALLOWED L.L. = 125 #/□'

WE FEEL THAT IT IS RIGHT TO USE THE 4" SPACING BECAUSE THE OTHER TWO FLOORS SHOW IT 4"

BEAM CHECK



CONSTANTS

$l = 14.42'$; $d = 20''$

$A_s = 2.46 \square''$ $b = 12 \times 3.65'$

$\frac{f_c}{f_s} = .0555$ ∴ N.A. IN FLANGE & BEAM NOT "T"

CHECK AS RECTANGULAR BEAM

MOMENT

$$\frac{12 \times w \times \overline{14.42}^2}{10} = 2.46 \times 18000 \times .867 \times 20$$

$$w = 3070\ \#/L.FT.$$

SHEAR (NO DOUBT THERE ARE STIRRUPS AS IN FLOOR ABOVE THO WE DID NOT SEE THEM)

$$V_c = 8 \times 20 \times .867 \times 50 = 6930\ \#$$

$$V_s = \frac{2 \times .11 \times 18000 \times .867 \times 20}{3} = 22900\ \#$$

$$V = 29830\ \#$$

$$w = \frac{2 \times 29830}{14.42} = 4140\ \#/LIN.\ FT.$$

D.L.

$$SLAB = \frac{6}{12} \times 10 \times 1 \times 150 = 750\ \#/LIN.FT.$$

$$BEAM = \frac{14}{12} \times \frac{8}{12} \times 1 \times 150 = 117\ \#/LIN.FT$$

TOTAL DEAD LOAD = 867 #/LIN. FOOT

∴ ALLOWED LIVE LOAD = 3070 − 867 = 2203 #/L.F.

$$= \frac{2203}{10} = 220\ \#/\square'$$

BOND

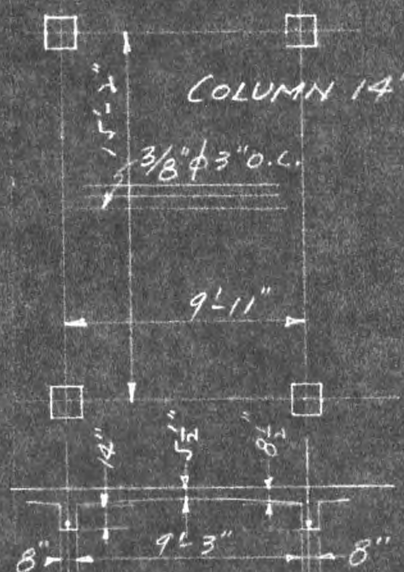
THO WE DO NOT KNOW THE STEEL IN THE BEAM OVER THE SUPPORT (COL.) WE ASSUME IT TO BE THE SAME AS IN THE POINT OPENED SINCE BOND WAS LITTLE UNDERSTOOD IN 1907.

$$\therefore \frac{14.42\ w}{2} = 7.86 \times 100 \times .867 \times 20$$

$$w = 1900 - 867 = 103\ \#/\square'$$

SECOND FLOOR

FLOOR CHECK



TYPICAL PANEL

CONSTANTS

$\ell = 9.25$; $A_s = (.405)\frac{\pi}{4} = .129^{\square}{}''$

$\Sigma_o = 1.28''$ $d = 5''$ AT ℄

$d = 8''$ AT BEAM; $M = \frac{w\ell^2}{10}$

MOMENT ℄

$$\frac{12\,w \times \overline{9.25}^2}{10} = 4 \times .129 \times .867 \times 5 \times 18000$$

$$w = 392\,\#/\square'$$

SHEAR

$$\frac{w \times 9.25}{2} = 12 \times 8 \times .867 \times 50$$

$$w = 900\,\#/\square'$$

BOND

$$\frac{w \times 9.25}{2} = 4 \times 1.28 \times 100 \times .867 \times 8$$

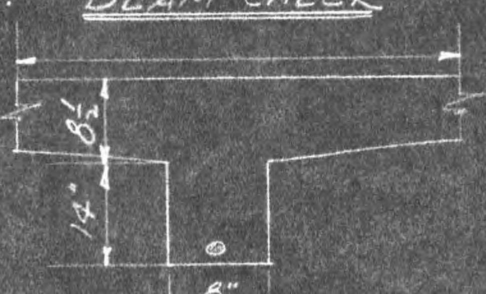
$$w = 770\,\#/\square'$$

DEAD LOAD

$$\frac{7}{12} \times 150 = 88\,\#/\square'$$

$\therefore$ ALLOWED L.L. $= 392 - 88 = 304\,\#/\square'$

BEAM CHECK



CONSTANTS

$\ell = 14.17'$; $d = 20\tfrac{1}{2}''$

$A_s = 1.23^{\square}{}''$; $b = 3.54'$

$\frac{f_c}{f_s} = .0555$ $\therefore$ N.A. IN FLANGE

CHECK AS RECTANGULAR BEAM

MOMENT

$$\frac{12 \times w \times 14.17^2}{10} = 1.23 \times 18000 \times .867 \times 20\tfrac{1}{2}$$

$$w = 1635\,\#/\text{LIN. FOOT}$$

SHEAR $3/8''\phi$ U STIRRUPS $3''$ O.C.

$$V_c = 8 \times 20.5 \times .867 \times 50 = 7100^{\#}$$

$$V_s = \frac{2 \times .11 \times 18000 \times .867 \times 20\frac{1}{2}}{3} = 23500^{\#}$$

$$V = 30600^{\#}$$

$$w = \frac{2 \times 30600}{14.17} = 4320^{\#}/\text{LIN. FT.}$$

BOND

$$\frac{w \times 14.17}{2} = 3.86 \times 100 \times .867 \times 20.5$$

$$w = 970^{\#}$$

DEAD LOAD

SLAB = $\frac{7}{12} \times 1 \times 9.92 \times 150$	=	$868^{\#}$/L.F.
BEAM = $\frac{14}{12} \times \frac{8}{12} \times 1 \times 150$	=	117
TOTAL D.L. =	=	$985^{\#}$/L.F.

FROM THIS WE SEE THAT THE OTHER BEAMS MUST HAVE $2-1\frac{1}{4}''\phi$ INSTEAD OF ONE $\therefore$ WE OBTAIN THE FOLLOWING

MOMENT	=	3270 − 985 =	$2285^{\#}$/L. FOOT
SHEAR	=	SAME	$4320^{\#}$/L.F.
BOND	=	1940 − 985 =	$955^{\#}$/L.F

$$\therefore \text{ALLOWED LOAD} = \frac{955}{9.92} = 96^{\#}/\square'$$

COLUMN CHECK

WE COULD NOT GET INTO COLUMN TO SURVEY THE STEEL BUT WE HAVE ASSUMED THAT THE MINIMUM WAS USED AND CHECKED IT ACCORDINGLY — NO NEED TO CHECK OTHERS.

BASEMENT COLUMN

MOMENT 50% ABOVE = 3300'#

LOAD .... 86000#

X_0 .... .92"

X_0/t .... .066

t/X_0 .... 15.2

$$\left\{\begin{array}{l} p \text{ MINIMUM} = .005 \\ f_c = .3 f_c' = 750\#/\square" \\ f_s = 16000\#/\square" \\ n = 12 \end{array}\right\}$$

$p(n-1)$ .... $.005 \times 11 = .055$

$\frac{d'}{t}$ .... .072 OR .143

$$\left.\begin{array}{l} \dfrac{f_c bt}{N} \text{ USING } \dfrac{d'}{t} = .072 = 1.29 \\ \dfrac{f_c bt}{N} \text{ USING } \dfrac{d'}{t} = .143 = 1.30 \end{array}\right\} \text{DIAGRAM \#4 (NOTES-F.C.S.)}$$

USING $b = t = 14"$

$$f_c = \frac{1.29 \times 86000}{14 \times 14} = 565\#/\square"$$

$$f_c = \frac{1.30 \times 86000}{14 \times 14} = 570\#/\square"$$

BOTH ARE QUITE SAFE SEE ALLOWED f_c

CONCENTRATED LOADS ON SECOND FLOOR
(SEE PLATES V & VI)

BEAM CHECK

BEAM UNDER TILE PARTITION WALL

D.L.

9.25' TILE WALL	= 9.25 × 15.5 × 20' =	2860
SLAB	= 5/12 × 9.25 × 14.5 × 150 =	9050
BEAM	= 1.17 × .67 × 14.5 × 150 =	1700
TOTAL		13610#

BENDING MOMENTS

$℄$ D.L. $= 13610 \times 14.5 \times \frac{1}{10} =$ 19700'#

$℄$ L.L. $= (4500 \times 7.75) - (2500 \times 5.75) - (2500 \times 1.75) =$ 16200'#

TOTAL 35900'#

$$f_c = \frac{2 \times 35900 \times 12}{.86 \times .4 \times 43.5 \times 18 \times 18} = 180\#/\square''$$

$$A_s \text{ (REQUIRED)} = \frac{35900 \times 12}{18000 \times .86 \times 18} = 1.57\square''$$

$$V = 6805 + 5000 = 11805\#$$

$$V_c = 40 \times 8 \times 18 \times .867 = 4950\#$$

$$V_s = V - V_c = 6845\#$$

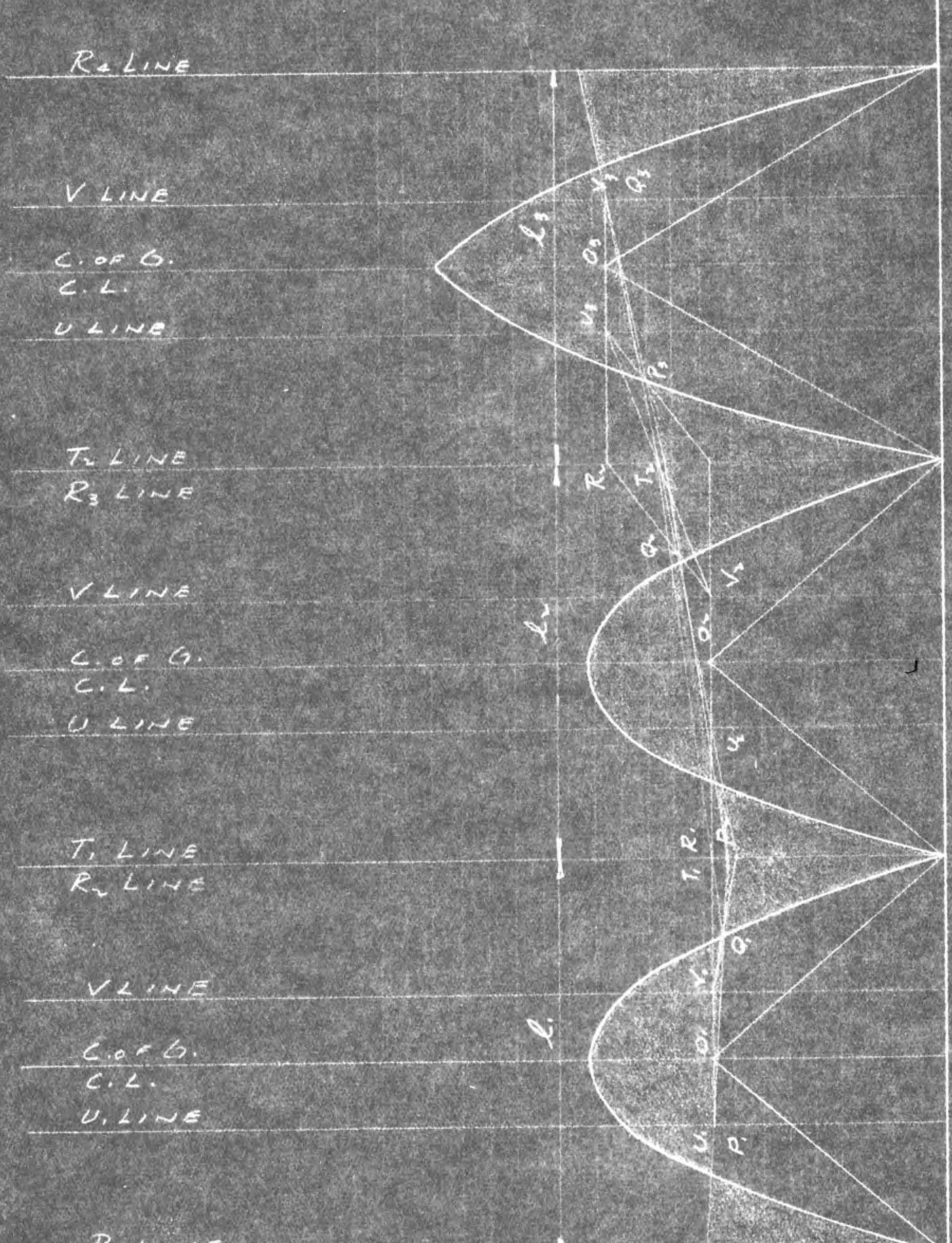
$$\therefore f_s = \frac{sV_s}{A_s j d} = \frac{3 \times 6845}{2 \times .11 \times .867 \times 18} = 6000\#/\square''$$

$$\Sigma_o = \frac{11805}{80 \times .867 \times 18} = 9.53''$$

MUST BE 2-1½"φ FOR SAFETY IN BOND

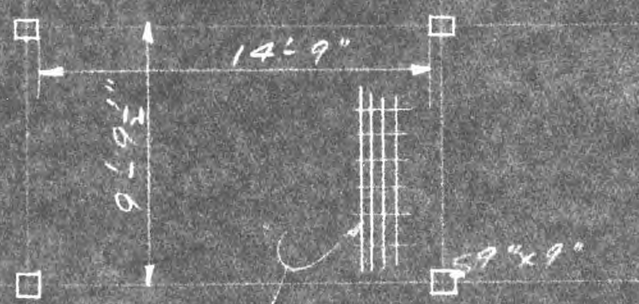
M_1 = MAX. ORDINATE = 26700'#. $\therefore \dfrac{A_1}{l_1} = \dfrac{2}{3}M_1 = 17800$

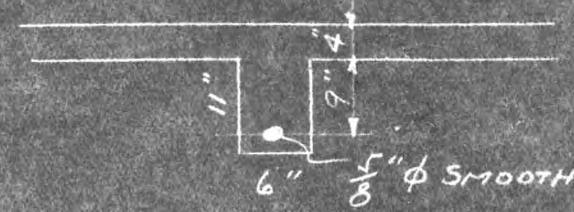
M_2 = MAX. ORDINATE = 37950'#. $\therefore \dfrac{A_3}{l_3} = \dfrac{2}{3}M_3 = 25300$



ROOF BEAM BENDING MOMENTS DUE TO HOIST FOURTH FL.

CHECK OF HOIST LOAD ON ROOF BEAM





"9 MESH MICROMETER DIA. = .153"

HOIST = $1\frac{1}{2}$ TON (3000#) AT CENTER LINE OF BEAM

UNIFORM LOAD

D.L.

SLAB $= \frac{4}{12} \times 1 \times 9.83 \times 150 =$ 492#

BEAM $= \frac{11}{12} \times 1 \times \frac{6}{12} \times 150 =$ 68#

1" GRAVEL & TAR $= \frac{1}{12} \times 108 \times 9.83 =$ 95

L.L.

$30 \times 9.83 =$ 295

TOTAL LOAD / LIN. FOOT 950#

MAX. ℄ B.M (SEE B.M. DIAGRAM) = 14000'#

MAX −B.M. = 27000'#

$b = \frac{14.75}{4} = 3.69' = 44.3"$; $A_s = .307\square"$; $t = 4"$; $d = 13"$

$$\frac{M}{f_s b t^2} = \frac{14000 \times 12}{18000 \times 44.3 \times 16} = .0124; \quad p℄ = \frac{.307}{44.3 \times 13} = .00053$$

NOT "T" BEAM

℄

$$f_s = \frac{14000 \times 12}{.307 \times .862 \times 13} = 48800\#/\square"$$

$$f_c = \frac{2 \times 12 \times 14000}{44.3 \times 169 \times .4 \times .867} = 130\#/\square"$$

SUPPORT

$$f_s = \frac{27000 \times 12}{.307 \times .862 \times 13} = 92500\#/\square"$$

$$f_c = \frac{2 \times 27000 \times 12}{6 \times 169 \times .4 \times .867} = 1880\#/\square"$$

OVER STRESSED

From these computations and the report included, we can readily see the inconsistencies encountered in checking a design of any structure of reinforced concrete built twenty years or more ago. We can also see the value of a historical background to such a survey for without it, we are not able to make assumptions consistent with the age of reinforced concrete design.

Many things of interest have shown up in our survey. One of much value is the good condition of the reinforcing steel in all beams opened up for inspection. In no place was rust seen any further advanced than as it is usually seen on steel before pouring.

We can infer that little was known or at least utilized about negative bending moments. This is shown in the fact that the top of nearly every beam has cracks, some quite dangerous and others just showing.

We observed further cracks at the column line perpendicular to the beams which must show the absence of enough temperature steel. Bending in the center line of the beam span would cause a tendency to put tension in the upper face of the slab at the column line and if steel were not present cracks most assuredly would occur.

The concrete is quite good in the beams but the slabs show poor construction joints and much sand. Evidently there was a shortage of cement(?) at the time but we have already remarked about inspection of engineering works.

Doesn't this survey give us a good understanding of the phrase, "Concrete for Permanence"? After approximately a quarter century we see a structure with all its faults in design, still as permanent in appearance as it was the day it was poured into shape.

Surely man's eternal material has been found - it was concrete they spoke of when they said "the eternal rocks".

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